DOE/OR/07-2038&D1

Seismic Investigation Report for Siting of a Potential On-Site CERCLA Waste Disposal Facility at the Paducah Gaseous Diffusion Plant Paducah, Kentucky

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CERTIFICATION

Document Identification: ***Transmittal--Seismic Investigation Report for Siting of a Potential On-Site CERCLA Waste Disposal Facility at the Paducah Gaseous Diffusion Plant, Paducah, Kentucky,*** **DOE/OR/07-2038&D1**

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons directly responsible for gathering the information, the information submitted is to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

U.S. Department of Energy (DOE)
Owner and Operator

W. Don Seaborg, Paducah Site Manager — 7/31/02
Date Signed

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons directly responsible for gathering the information, the information submitted is to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Bechtel Jacobs Company LLC
Co-operator

Gordon L. Dover, Paducah Manager of Projects — 7/31/02
Date Signed

BECHTEL JACOBS
Bechtel Jacobs Company LLC

DOE Contract No. DE-AC05-98OR22700
Job No. 23900
August 2, 2002

Mr. W. Don Seaborg
Paducah Site Manager
U.S. Department of Energy
P.O. Box 1410
Paducah, KY 42002-1410

Subject: Transmittal--*Seismic Investigation Report for Siting of a Potential On-Site CERCLA Waste Disposal Facility at the Paducah Gaseous Diffusion Plant, Paducah, Kentucky,* DOE/OR/07-2038&D1

Dear Mr. Seaborg:

Enclosed are 19 copies of the subject document. This report summarizes and presents conclusions from a regional and site-specific seismic investigation at the Paducah Gaseous Diffusion Plant (PGDP), Paducah, Kentucky. This investigation was performed to characterize a portion of Department of Energy property that is under consideration for potentially siting a disposal facility for wastes generated from future environmental restoration activities implemented under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 at PGDP.

Please forward 11 copies of the subject document to the following at the Commonwealth of Kentucky regulatory agencies: Ms. Gaye Brewer (1), Mr. Rob Daniel (7), Ms. Janet Miller (1), and Dr. Eric Scott (2). Four copies are to be transmitted to Mr. Jeff Crane of the U.S. Environmental Protection Agency. The remaining four copies are for your use.

Suggested text for your use in transmitting the documents to the regulatory agencies is also enclosed. Distribution is being made in accordance with the *Standard Distribution List for Bechtel Jacobs Company LLC Primary and Secondary Documents (4/29/02).*

If you have any questions or need additional information, please contact Jim Skridulis of my staff at 5056.

Sincerely,

Gordon L. Dover
Paducah Manager of Projects

GLD:dj
LTR-PAD/EP-DJ-02-0092

Enclosures: 1. Subject document
2. Suggested text

DOE/OR/07-2038&D1

Seismic Investigation Report for Siting of a Potential On-Site CERCLA Waste Disposal Facility at the Paducah Gaseous Diffusion Plant Paducah, Kentucky

Date Issued—August 2002

Prepared for the
U.S. Department of Energy
Office of Environmental Management

Environmental Management Activities at the
Paducah Gaseous Diffusion Plant
Paducah, Kentucky 42001
managed by
Bechtel Jacobs Company LLC
for the
U.S. DEPARTMENT OF ENERGY
under contract DE-AC05-98OR22700

Prepared by
SAIC Engineering, Inc.
151 Lafayette Drive
Oak Ridge, Tennessee 37830

I, the undersigned, certify that I am a qualified professional engineer with competence in the subject matter dealt with in this document. I further certify that this document has been prepared under my responsible charge according to my knowledge, information, and belief in accordance with applicable standards of practice, the laws and rules governing the engineering profession of the Kentucky State Board of Licensure for Professional Engineers under KRS 322.00.

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contributed to the preparation of this document and should not be considered an eligible contractor for its review.

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ACRONYMS

ASTM	American Society for Testing and Materials
bgs	below ground surface
BP	years before present, where "present" is defined as 1950 A.D.
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act of 1980
C_N	correction factor
CRR	cyclic resistance ratio
CSR	cyclic stress ratio
DOE	U.S. Department of Energy
DPT	direct push technology
EPA	U.S. Environmental Protection Agency
FFA	Federal Facility Agreement
fps	feet per second
GPR	ground penetrating radar
ISGS	Illinois State Geological Survey
KDWM	Kentucky Division of Waste Management
msl	mean sea level
NEPA	National Environmental Protection Act
NMSZ	New Madrid seismic zone
NPL	National Priorities List
NRC	Nuclear Regulatory Commission
pcf	pounds per cubic foot
PGA	peak ground acceleration
PGDP	Paducah Gaseous Diffusion Plant
PSA	peak spectral acceleration
PSHA	probabilistic seismic hazard analysis
REI	Risk Engineering, Inc.
RGA	Regional Gravel Aquifer
SCPT	seismic cone penetrometer test
SPT	Standard Penetration Test
tsf	ton per square foot
UCRS	Upper Continental Recharge System
UHS	uniform hazard spectra
USCS	Unified Soil Classification System
WKWMA	West Kentucky Wildlife Management Area
WVSZ	Wabash Valley seismic zone

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EXECUTIVE SUMMARY

INTRODUCTION

This Seismic Investigation report has been prepared to summarize and present conclusions from a regional and site-specific Seismic Investigation at the Paducah Gaseous Diffusion Plant (PGDP), Paducah, Kentucky. This investigation has been performed to characterize a portion of DOE property that is under consideration for potentially siting a disposal facility for wastes generated from future environmental restoration activities implemented under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) at PGDP. PGDP was placed on the National Priorities List (NPL) in May 1994. In 1998, the U.S. Department of Energy (DOE), the U.S. Environmental Protection Agency (EPA), and the Commonwealth of Kentucky entered into a Federal Facility Agreement.

The Project Core Team (consisting of representatives from DOE, EPA, and the Commonwealth of Kentucky) has been evaluating options for disposing of future CERCLA wastes. One option under consideration is the disposal of those wastes in a potential on-site facility. A Siting Study was initiated in 2000 to support DOE's selection of a site for such a facility. Site 3A, located south of the plant, is a reconfiguration of one such site as presented in the Siting Study. The Seismic Investigation program consists of field characterization at PGDP, specifically at Site 3A, to determine whether it is feasible as a candidate site.

The Project Core Team developed the following seven questions that, when answered, would fully address the seismic issues at Site 3A. Table ES.1 repeats these questions and presents a summary of the answers developed during the Seismic Investigation.

1. Is there evidence of paleoliquefaction?
2. Is there paleoseismic evidence of local strong motion?
3. Is there potential for future liquefaction?
4. Is there evidence of Holocene displacement of faults at PGDP?
5. Are there faults underlying the potential disposal facility site?
6. What is the peak ground acceleration (PGA) at the potential disposal facility site?
7. What are the characteristics of the design ground motion?

The Seismic Investigation program includes three primary tasks: (1) a Paleoliquefaction Study, (2) a Fault Study, and (3) a Geotechnical Study. Field activities were completed between September 28, 2001, and March 27, 2002. The following sections summarize the investigation activities completed and the conclusions reached in each study.

RESULTS OF THE PALEOLIQUEFACTION STUDY

The Paleoliquefaction Study was developed to address Questions 1 and 2, and to support answering Questions 3, 6, and 7. The study included a review of historical information on liquefaction in the region, a search for evidence of paleoliquefaction features in the region, an evaluation of borehole cores taken from Site 3A for evidence of past liquefaction, and an evaluation of the results of laboratory testing of soil samples collected from Site 3A to assess liquefaction potential. Paleoliquefaction is defined here as seismically induced liquefaction features associated with prehistoric Holocene or late Pleistocene earthquakes.

Table ES.1. Summary answers to Project Core Team questions to address seismic issues at Site 3A.

Question	Summary answer
1. Is there evidence of paleoliquefaction at or near PGDP?	Field observations made along the Ohio River in the vicinity of PGDP found no large liquefaction features. Smaller scale paleoliquefaction features may have been present but remained unobserved because of their relatively small size or veneer of river deposits and vegetative cover. There is no definitive evidence of paleoliquefaction at PGDP based on results of field investigations conducted along portions of Bayou and Little Bayou Creeks. The literature does report some small liquefaction features located along the banks of the Ohio River, about 8 miles northeast of PGDP, and along the Post Creek Cutoff, about 12 miles northwest of PGDP.
2. Is there paleoseismic evidence of local strong ground motion?	The absence of large paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. The small liquefaction features that have been reported in the literature are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.
3. Is there potential for future liquefaction at Site 3A?	Many of the soils present at the site are clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet the criteria that distinguish those fine-grained soils that could experience large-scale strain, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. Some liquefaction within the sands and deformation within the silts and clays could occur at PGAs approaching 0.5 g.
4. Is there evidence of Holocene displacement of faults at PGDP?	This study did not find Holocene displacement of faults at Site 3A. Several faults identified in seismic reflection data at Site 3A have been confirmed to extend through the Porters Creek Clay and into the materials underlying the surficial loess deposits. Three of these faults are interpreted to extend to within approximately 20 ft of the ground surface. One DPT borehole encountered three fault planes at depths between 22 ft and 28 ft. No faults were observed in the overlying loess. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age with ^{14}C dates ranging from 13,500 to 15,600 years BP. At the Barnes Creek site located 11 miles northeast of PGDP, this study found Holocene age displacement of faults in deposits with ^{14}C dates ranging from 5000 to 7000 years BP.
5. Are there faults underlying the potential disposal facility site?	The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults beneath Site 3A, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks. Several of the faults extend through the Porters Creek Clay and into the materials underlying the surficial loess. Three of these faults extend to within 20 ft of the ground surface.
6. What is the PGA at the potential disposal facility site?	Based upon data collected from Site 3A, the PGA at Site 3A is calculated to be 0.48 g for a 2500-year return period earthquake.
7. What are the characteristics of the design ground motion?	The design ground motions at Site 3A would be the same as those presented in a 1999 study performed by Risk Engineering, Inc. The shear-wave velocities in the soil column at Site 3A are similar to those determined previously at other locations on the DOE property, resulting in similar design ground motions.

BP = years before present, where "present" is defined as 1950 A.D.
DPT = direct push technology
PGA = peak ground acceleration
PGDP = Paducah Gaseous Diffusion Plant
REI = Risk Engineering, Inc.

The purpose of the Paleoliquefaction Study is to determine (1) the existence of liquefaction features in Quaternary-age deposits in the PGDP region and (2) whether this liquefaction, if found, is the result of past New Madrid-type earthquakes or local earthquakes that originated in the PGDP vicinity. The regional study was conducted within a 15-mile radius of PGDP. The study consisted of reviewing historical data and conducting a field survey, which included ground inspections of target streams. The ground inspections consisted of surveying the banks of the Ohio River, Mayfield Creek, Bayou and Little Bayou Creeks, and a limited number of private land areas. Fifteen priority sites were identified for further study along the Ohio River and creeks in southern Illinois.

Field investigations conducted as part of the Seismic Investigation found no large liquefaction features along the Ohio River in the vicinity of PGDP. The riverbank afforded adequate exposure of the sediments such that if large liquefaction features were present they should have been obvious. Smaller-scale paleoliquefaction features may have been present but were not observed because of their relatively small size or the typical veneer of river deposits and vegetative cover.

Field investigations conducted along portions of Bayou and Little Bayou Creeks found no definitive evidence of paleoliquefaction at PGDP.

The literature does report some small liquefaction features within 15 miles of PGDP. The closest are located along the banks of the Ohio River, about 8 miles to the northeast. These features are in the general vicinity of Fort Massac, Illinois, a location where liquefaction was reported during the February 7, 1812, New Madrid earthquake. These features were small and relatively unweathered, suggesting that they were probably outlying liquefaction features resulting from the 1811 and 1812 New Madrid earthquakes. Small liquefaction features are also reported in the literature along the Post Creek Cutoff, about 12 miles northwest of PGDP.

The absence of large paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. In this context "local strong ground motion" is defined as strong ground motion resulting from a local earthquake. The small liquefaction features that have been reported in the literature are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.

The site-specific evaluation consisted of evaluating data collected during the Geotechnical Study for liquefaction potential at Site 3A. Many of the soils present at the site are fine-grained clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet criteria that distinguish those fine-grained soils that could experience large-scale strain, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. However, based on calculations presented in this report, it was concluded that some liquefaction within the sands and deformation within the silts and clays could occur at a PGA approaching 0.5 g.

RESULTS OF THE FAULT STUDY

The purpose of the Fault Study is to determine whether Holocene-age faulting has occurred in the PGDP vicinity. The Fault Study is to answer Questions 4 and 5 posed by the Project Core Team and to assist in any subsequent facility design activities. The Fault Study included both regional and site-specific components.

Results of the Regional Fault Study

The regional Fault Study collected data to support the design of a potential on-site CERCLA waste disposal facility. Such data include displacement, earthquake magnitude, recurrence interval, and age of the most recent event. The regional Fault Study was conducted at a site in Massac County, Illinois, at/near Barnes Creek, which is located approximately 11 miles northeast of PGDP. Tasks completed at the Barnes Creek site included visual observations and measurements of features in the banks of Barnes Creek, a ground penetrating radar (GPR) calibration survey, and follow-up GPR survey and direct push technology (DPT) boreholes in the target fault area. These tasks were implemented to identify the key geologic units, their relationship with observed faults, and their dates of deposition to establish ages of past fault movements. Although excavation of test pits and a trench was originally planned to collect visible evidence of shallow faulting, data collected from the creek banks and DPT boreholes were sufficient in dating of the deposits and in determining that no correlation exists between the topography and faulting. Therefore, the DOE investigation team decided not to perform the test pits and trench excavation.

The bank study was conducted along an approximately 2600-ft-long portion of Barnes Creek. Visible faulting and other geologic features were studied at 12 locations. Fourteen organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

The GPR survey was conducted across the suspected location of a terrace graben, approximately 1100 ft north of Barnes Creek. Three parallel 900-ft lines were surveyed using a 200 MHz antenna; the survey provided high-resolution data of the uppermost sediments and was used to identify areas where the DPT boreholes should be located.

The DPT survey was conducted across the terrace graben area to identify potential Holocene faults, displacement at shallow depths, and surface morphology. Ten DPT boreholes were driven across a 450-ft section of the middle GPR survey line. The depths of the DPT boreholes varied between 32 and 63 ft, producing a total of nearly 404 ft of continuous core from the ten boreholes. Six organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

Geologic structures observed in Barnes Creek included individual joints, faults, clay dikes, and paired faults forming down-dropped blocks known as grabens. Neotectonic studies were carried out in a portion of Barnes Creek to determine if mapped faults have moved within the Holocene Epoch (within the last 10,000 to 12,000 years). Investigations in the creek identified five geologic units. The three oldest units, the Cretaceous McNairy Formation, and the gravels, sands, and silts of both the upper and lower Metropolis Formation, exhibit faults, clay dikes, and joints. The two youngest units, a surficial light brown sandy alluvium and an underlying light gray alluvium, did not exhibit faulting.

The trends (generally northeast-southwest) of the geologic structures in the oldest units and style of deformation is consistent with bedrock faults mapped to the north of the study area by the Illinois State Geological Survey (ISGS). The northeast-southwest trends are also consistent with the trend of the New Madrid seismic zone to the southwest, suggesting that these features may be related.

The relative timing of the observed deformations in the geologic structures varies. A number of geologic structures are limited to the McNairy Formation and clearly pre-date deposition of the Metropolis materials. Other features involve both the McNairy and Metropolis materials to the same extent, while others appear to be re-activation of old features in the McNairy after or during deposition of the Metropolis materials.

Radiocarbon ages confirm that repeated deformation has occurred along some of the observed faults. Deformation began prior to the deposition of the lower Metropolis (late Pleistocene), continued during the deposition of the upper Metropolis (which is 5000 to 7000 years old), and most recently occurred in the mid-Holocene, after the deposition of the upper Metropolis (within the last 5000 years). Therefore, faults observed at the Barnes Creek site did extend into Holocene-age deposits. The maximum displacement observed in a single event is approximately 1 ft in the lower Metropolis.

Investigation of the terrace graben area concluded that the observed stratigraphy is consistent with a combination of two models: (1) a graben with up to 50 ft of displacement within the past 12,000 years, and (2) an erosional feature with up to 50 ft of infilling within the past 12,000 years. Radiocarbon ages in the terrace graben area at the Barnes Creek site indicate that the deep fine-grained sediments beneath the Metropolis are approximately 11,000 years old, indicating that the overlying Metropolis dates from the late Pleistocene or early Holocene.

Results of the Site-Specific Fault Study

The site-specific Fault Study at Site 3A was developed to answer Questions 4 and 5. The site-specific Fault Study was conducted to determine whether evidence of Holocene faulting exists at Site 3A. Because Site 3A is entirely underlain by the Porters Creek Clay, initial activities were conducted to determine whether deformation of the Porters Creek Clay is apparent, based on results of a seismic compression wave (p-wave) reflection survey. Follow-up activities were then conducted to provide higher resolution data in order to determine whether displacement is apparent at relatively shallow depths. These follow-up activities included a seismic horizontal shear wave (s-wave) reflection survey and DPT boreholes within a target fault zone at Site 3A. Although excavation of test pits and a trench was originally planned so as to collect visible evidence of shallow faulting, field conditions (e.g., water levels, excessive excavation depths, obstructions, and wetlands) were not amenable to excavation nor to successful data collection. Therefore, the DOE investigation team decided not to perform the test pits and trench excavation.

The initial p-wave survey evaluated four combinations of energy sources and selected the T-15000 iVi Minivib as providing the highest resolution at Site 3A. Approximately 16,000 lin. ft of survey data were collected along five lines (seven segments) using a geophone group interval of 5 ft, shot spacing of 10 ft, 144-channel, 36-fold survey configuration. Several horizons were successfully imaged beneath Site 3A, including the top of limestone bedrock, the McNairy Formation (lower sand facies), and portions of the Porters Creek Clay. The results indicated potential young faults extending from the limestone bedrock up into the Porters Creek Clay.

A GPR calibration survey was conducted to determine whether GPR was capable of penetrating local clays and silts to identify subsurface features. At the Barnes Creek site, four GPR tests were conducted using 200, 100, 80, and 16 MHz antennas along a 1500-ft test line. The 200 MHz antenna was selected as providing the greatest resolution at the Barnes Creek site. At Site 3A, two GPR tests were conducted using 200 and 40 MHz antennas along a 750-ft test line. Because neither of these antennas provided suitable resolution of the geology at Site 3A, no follow-up GPR survey was recommended for Site 3A.

The follow-up s-wave survey focused on two areas at Site 3A where potential young faults were suggested by the p-wave survey. Approximately 2300 lin. ft of data were collected along two lines using a MicroVib source, 96-channel seismograph, 48-fold survey, and 40-Hz horizontal component geophones at a group interval of 2 ft and shot spacing of 2 ft. Several horizons were successfully imaged, including the Porters Creek Clay, an overlying firm sand unit, and portions of the loess. Several potential faults extending up to or near the bottom of the loess unit were identified.

Ten DPT boreholes were driven along the two s-wave survey lines to depths ranging from 21 to 40 ft. In addition, an 11th DPT borehole was driven at one of the planned shallow boring locations (SB-04). The DPT survey produced a total of nearly 400 ft of continuous core from the 11 boreholes. Three fault planes were observed at depths of 22 to 28 ft in a DPT borehole near the southern boundary of Site 3A. Five organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks.

Several of the faults identified in the p-wave survey extend through the Porters Creek Clay at an approximate depth of 30 to 60 ft and into the materials underlying the surficial loess deposits. Three of these faults extend to within approximately 20 ft of the ground surface. A DPT borehole drilled adjacent to one of the postulated shallow faults encountered three fault planes at depths between 22 and 28 ft. No faults were observed in the overlying loess sampled in this same DPT borehole. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age, with ^{14}C dates ranging from about 13,500 to 15,600 years before present, where "present" is defined as 1950 A.D. (BP).

Therefore, this study did not find Holocene displacement of faults at Site 3A.

RESULTS OF THE GEOTECHNICAL STUDY

The purpose of the Geotechnical Study is to determine the variability of the lithology underlying Site 3A and to acquire seismic and geotechnical characteristics of the deposits at Site 3A for use in the design of a potential on-site CERCLA waste disposal facility. The Geotechnical Study is to provide data that support answers to Questions 3, 6, and 7 posed by the Project Core Team. Field activities included drilling, sampling, and testing of two deep boreholes, one using a Rotosonic drilling technique (DB-01) and an adjacent one using a mud rotary drilling technique (DB-02). The activities also included drilling, sampling, and testing of shallow mud rotary boreholes and seismic cone penetrometer test (SCPT) soundings.

The deep Rotosonic borehole (DB-01) was drilled to a total depth of 359 ft, producing a continuous core. A downhole natural gamma log of the borehole was conducted. The deep mud rotary borehole (DB-02) was drilled to bedrock, which was encountered at a total depth of approximately 400 ft. Standard Penetration Test (SPT) sampling was conducted continuously to a depth of 75 ft, and at approximately 20-ft intervals thereafter to 186 ft. A downhole seismic velocity log of the borehole was conducted.

Five mud rotary boreholes were drilled to depths ranging from 52 to 70 ft. Two additional boreholes were planned; however, because heavy rainfall prevented access by the drill rig, the DOE investigation team converted the planned borings into one DPT borehole (SB-04) and one SCPT sounding (SB-07) so that a lighter track-mounted rig could be used. SPT sampling was conducted continuously throughout the depth of the boreholes. Three organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

Forty-four Shelby tube samples and 153 split spoon samples were collected from the borings and sent to an off-site geotechnical laboratory for analysis. Forty of the Shelby tube samples were analyzed for in-place density, vertical permeability, triaxial compressive strength, and one-dimensional consolidation. Forty-eight split spoon samples were analyzed for index properties and contaminant transport properties.

Fourteen SCPT soundings were completed at 11 locations at Site 3A to depths ranging from 10 ft (refusal) to 70 ft, for a total of 623 ft. Continuous tip, sleeve, and pore pressure measurements were collected from 6 ft to total depth in each SCPT sounding. Twenty-nine pore pressure dissipation tests were conducted in varying lithologies. Seismic shear wave velocities were measured at approximate 3-ft intervals.

Bedrock was encountered at a depth of 400 ft below ground surface (bgs) in borehole DB-02 at Site 3A. The McNairy Formation was encountered overlying bedrock to a depth of 155 ft bgs, for a total thickness of 245 ft. The Porters Creek Clay was encountered overlying the McNairy to a depth varying between 30 and 60 ft bgs. Terrace Deposits typically overlay the Porters Creek Clay to a depth of 15 to 20 ft bgs. Surficial loess deposits were encountered overlying the Terrace Deposits. In some areas there are younger alluvial deposits of Holocene age that fill former erosional features incised in the loess.

Results of settlement calculations predict that the total settlement of a potential disposal cell constructed to a height of 102 ft above ground surface would result in more than 5 ft of settlement in the center of the cell area. Differential settlement may be as large as 2 to 3 ft across the disposal cell. Detailed design would need to account for such differential settlement by increasing the slopes of the base grades, bottom liner, and drain lines, and by selecting appropriate construction materials. It should be noted that the amount of disposal cell settlement may be overestimated because of difficulties in retrieving undisturbed samples in the Porters Creek Clay. Settlement would occur relatively rapidly, with 90% of the settlement occurring in less than 2 years of fill placement so that settlement would be essentially completed by the time the cell is filled.

Results of bearing capacity analysis indicate that the bearing capacity of the foundation soils is adequate to support a potential CERCLA waste disposal facility at Site 3A.

RESULTS OF THE SEISMIC DESIGN MODEL

A seismic design model was developed for Site 3A to answer Questions 6 and 7; namely, to determine the PGA and design ground motions for a potential on-site CERCLA waste disposal facility. The model was developed based on the data collected during the site-specific Fault Study and Geotechnical Study.

A probabilistic seismic hazard analysis was performed to determine what the PGA and other related ground motions (ground shaking frequency, velocity, and displacements) would be at Site 3A for an earthquake having a 2500-year return period. The corresponding PGA value at the top of rock (400 ft deep) was determined to be 0.71 g.

Because the potential on-site CERCLA waste disposal facility would be founded on soil materials up to 400 ft thick, further analysis was needed to calculate the PGA at the top of the soil (at the base of the disposal cell). Soils typically amplify the top-of-rock PGA; therefore, a factor, called the "soil amplification factor," is used to convert top-of-rock PGA to top-of-soil PGA. A site-specific soil amplification factor was calculated for Site 3A based on the shear-wave velocities measured in the deep borehole (DB-02) and SCPT soundings using the methodology employed by Risk Engineering, Inc. in its 1999 study. The soil amplification factor for a top-of-rock PGA of 0.71 g was calculated to be 0.67. This results in a top-of-soil PGA of 0.48 g for a 2500-year return period earthquake at Site 3A.

This value is equal to the top-of-soil PGA value of 0.48 g interpolated from Risk Engineering, Inc. in a previous reevaluation study at PGDP. Therefore, the recommended top-of-soil PGA for the design of a potential CERCLA waste disposal facility at Site 3A is 0.48 g.

The shear-wave velocities in the soil column at Site 3A are similar to those determined previously at other locations on the DOE property, resulting in similar design ground motions. Therefore, the design ground motions at Site 3A would be the same as those determined by Risk Engineering, Inc. A uniform hazard spectra that relates ground acceleration to the ground shaking frequency is presented in this report to define the design ground motions at Site 3A.

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1. INTRODUCTION

This Seismic Investigation report has been prepared to summarize and present conclusions from a regional and site-specific Seismic Investigation at the Paducah Gaseous Diffusion Plant (PGDP), which is located approximately 10 miles west of the city of Paducah, Kentucky (Fig. 1.1). This investigation was performed to characterize a portion of the U.S. Department of Energy (DOE) property that is under consideration for potentially siting a disposal facility for wastes generated from future environmental restoration activities implemented under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) at PGDP.

PGDP was placed on the National Priorities List (NPL) in May 1994. In 1998, DOE, the U.S. Environmental Protection Agency (EPA), and the Commonwealth of Kentucky entered into a Federal Facility Agreement (FFA), which is the interagency agreement governing the cleanup of the facility pursuant to CERCLA Sect. 9620(e)(2).

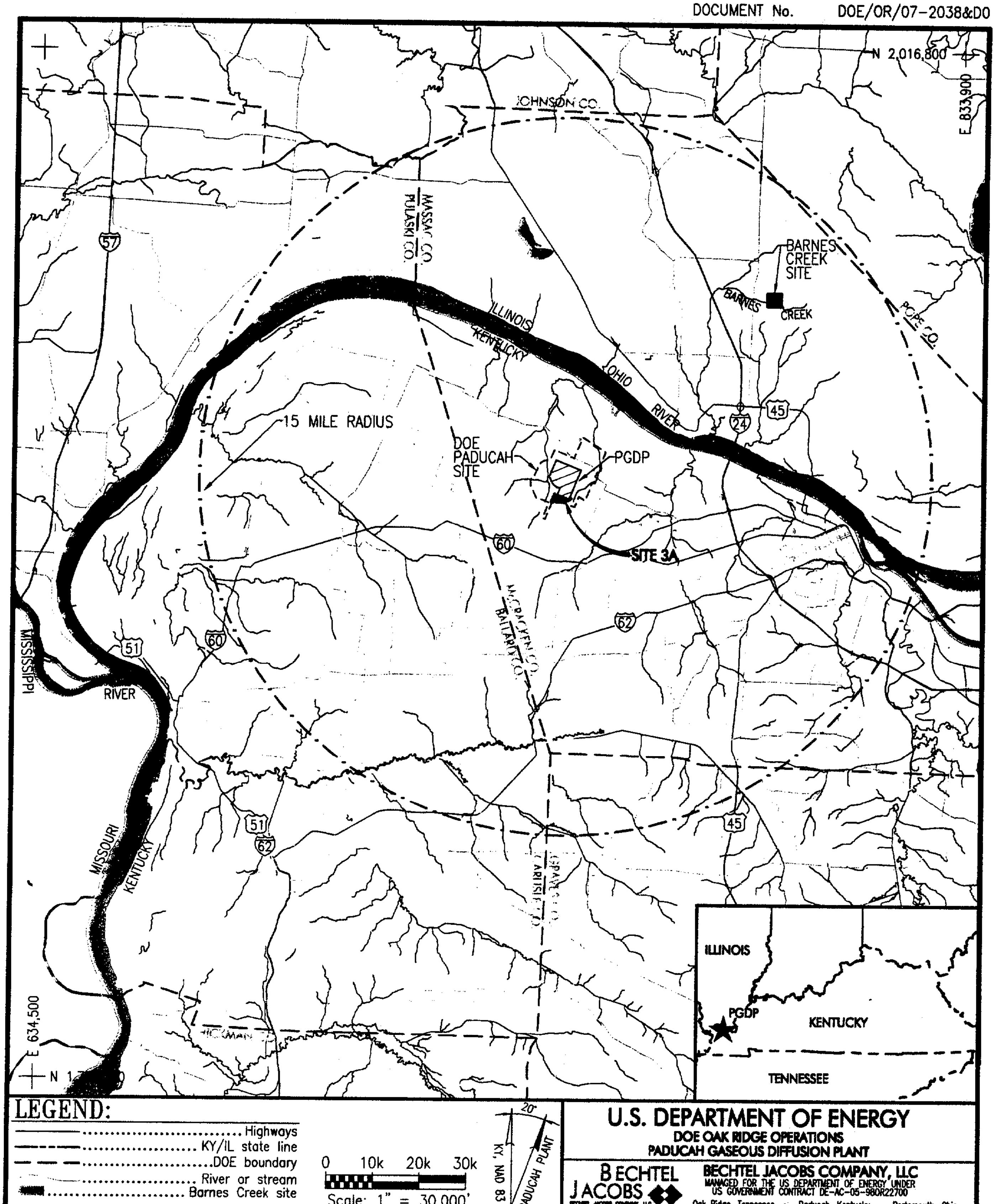
DOE is evaluating options for the disposition of waste materials that are anticipated to be generated as a result of future CERCLA actions. One option under consideration is the disposal of those wastes in a potential on-site facility. A Siting Study was initiated in 2000 to support DOE's selection of a site for such a facility, if on-site disposal were to be selected as the preferred alternative for the CERCLA waste. The Siting Study, summarized in *Identification and Screening of Candidate Sites for a Potential CERCLA Waste Disposal Facility at the PGDP, Paducah, Kentucky*, DOE/OR/07-1939&D1, identified three sites for consideration (DOE 2001). During their May 17, 2001, meeting, the Site-Specific Advisory Board (currently the Citizens Advisory Board) recommended that DOE focus investigation efforts at a fourth site, Site 3A. Site 3A is a reconfiguration of Site 3 as presented in the Siting Study, located south of the PGDP security fence, north and east of the intersection of Hobbs Road and Dyke Road (Fig. 1.2).

The Seismic Investigation consists of site-specific field characterization at Site 3A to determine whether it is feasible as the final candidate site. The Seismic Investigation also includes characterization of regional features with specific emphasis on the Barnes Creek site located in Massac County, Illinois. If Site 3A is determined to be feasible, the results of this field characterization would be considered in the evaluation of any waste disposal options.

1.1 BACKGROUND

On April 5, 2001, the Project Core Team (consisting of representatives from DOE, EPA, and Commonwealth of Kentucky) met to develop a field investigation program to address seismic issues associated with potentially siting a CERCLA waste disposal facility at PGDP. The Project Core Team identified the seismic issues, discussed likely resolution strategies, and outlined field activities that could be performed to address the issues. The Team developed a list of questions that, when answered, would fully address the seismic issues. The Team developed the following questions:

1. Is there evidence of paleoliquefaction?
2. Is there paleo-seismic evidence of local strong motion?
3. Is there potential for future liquefaction?
4. Is there evidence of Holocene displacement of faults at PGDP?
5. Are there faults underlying the potential disposal facility site?
6. What is the peak ground acceleration (PGA) at the potential disposal facility site?
7. What are the characteristics of the design ground motion?



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PADUCAH GASEOUS DIFFUSION PLANT

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MANAGED FOR THE US DEPARTMENT OF ENERGY UNDER US GOVERNMENT CONTRACT DE-AC-05-98OR22700
Oak Ridge, Tennessee • Paducah, Kentucky • Portsmouth, Ohio

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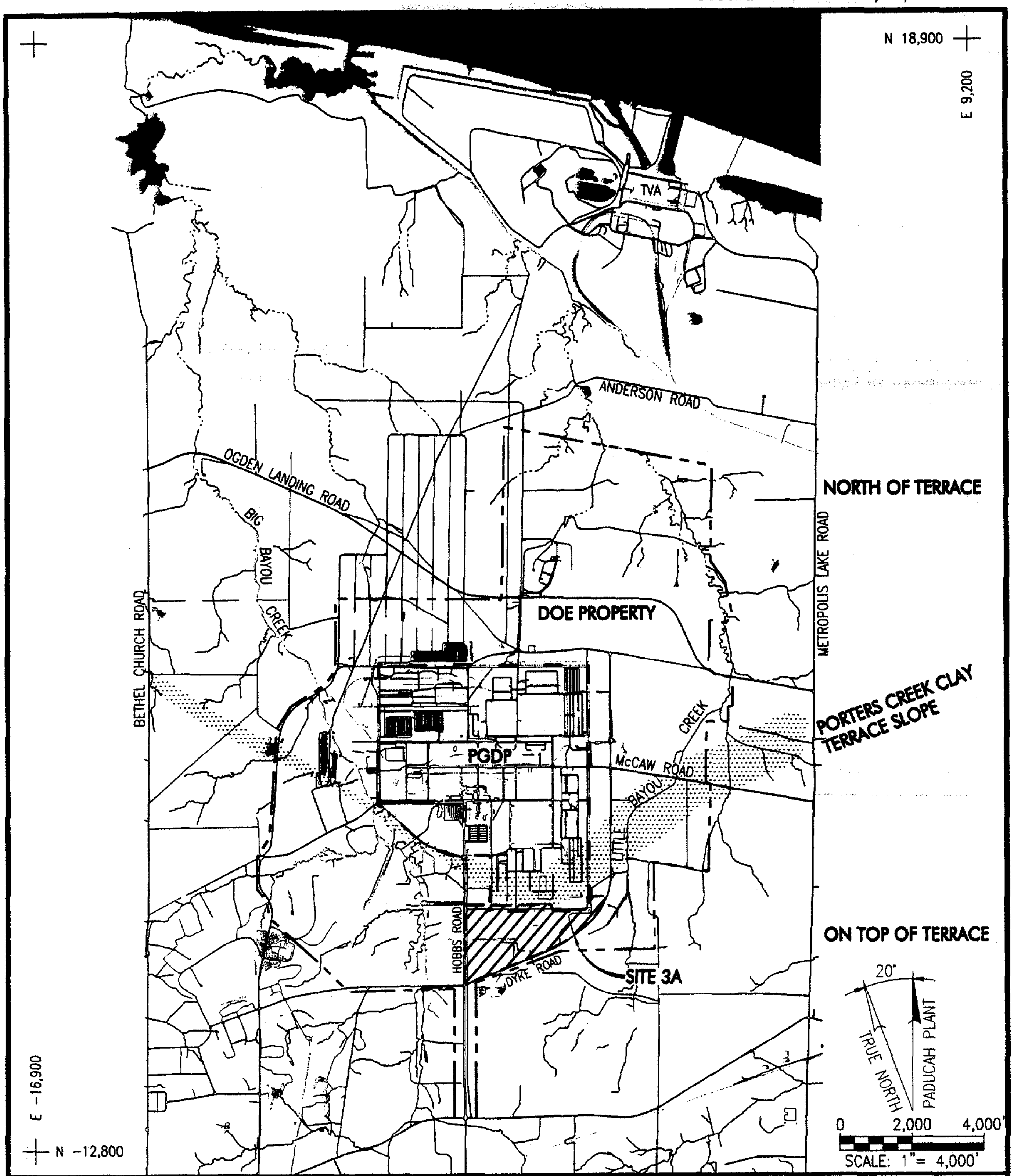
Fig. 1.1. Location of PGDP and regional sites.

Figure No. /99049/DWGS/P34VICN83
DATE 07-08-02



LEGEND:
- .... DOE property boundary
- ....PGDP boundary
- ....Asphalt road
- ....Stream
- ....Site 3A
- ... Porters Creek Clay terrace slope

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Fig. 1.2. Location of Site 3A at PGDP.

Figure No. /99049/DWGS/P34LOC
DATE 07-10-02

The Team identified a single decision point in determining whether to continue the effort to site a potential CERCLA waste disposal facility: the confirmation of active (Holocene-age) faulting at or near PGDP. If the study identified such faulting, then DOE could decide to discontinue the investigation. A fault is defined as "active" by applicable federal and Commonwealth of Kentucky regulations if it has had movement in the last 10,000 to 12,000 years (during the Holocene Epoch). The Team also agreed that "to be considered" criteria would include two other definitions of active faulting: (1) Nuclear Regulatory Commission (NRC) criteria (movement in the last 35,000 years or repetitive movement on a single fault in the last 500,000 years) and (2) U.S. Army Corps of Engineers criteria (movement in the last 100,000 years).

1.2 PURPOSE OF THE SEISMIC INVESTIGATION PROGRAM

The purpose of the Seismic Investigation program is to develop a better understanding of the seismic conditions at PGDP. The Seismic Investigation program includes three primary tasks: (1) Paleoliquefaction Study, (2) Fault Study, and (3) Geotechnical Study. The workplan for conducting the Seismic Investigation is presented in the *Seismic Assessment Plan for Siting of a Potential On-Site CERCLA Waste Disposal Facility at the PGDP, Paducah, Kentucky*, BJC/PAD-207 (2001c), and the evaluations of National Environmental Protection Act (NEPA) values (BJC 2001a, 2001b, 2002a, and 2002b). The geotechnical study was identified as the Acquisition of Seismic and Geotechnical Design Data in the workplan.

The purpose of the Paleoliquefaction Study is to determine (1) the existence of liquefaction features in Quaternary-age deposits in the PGDP region and (2) whether this liquefaction, if found, is the result of past New Madrid-type earthquakes or local earthquakes that originated in the PGDP vicinity. The Paleoliquefaction Study was structured to answer Questions 1 and 2 and support answering Questions 3, 6, and 7 posed by the Project Core Team. The Paleoliquefaction Study was conducted within a 15-mi radius of PGDP (Fig. 1.1). The study consisted of reviewing historical data and conducting a field survey, which included ground inspections of target streams and the Ohio River bank.

The Fault Study is comprised of two components: a regional Fault Study and a site-specific Fault Study. Each component has a different, but similar, purpose and each component addresses a different question. The purpose of the regional Fault Study is to determine if faults or postulated faults in the PGDP vicinity have experienced Holocene-age displacement. This study was developed to support answering Question 4 posed by the Project Core Team. During a scoping meeting held on June 6–7, 2001, DOE, EPA and the Commonwealth of Kentucky agreed that the purpose of the regional Fault Study is to collect data that would support designing a potential on-site CERCLA waste disposal facility (e.g., displacement, earthquake magnitude, recurrence interval, and age of the most recent event). DOE held a meeting (i.e., a "Target Fault Workshop") with EPA and the Commonwealth of Kentucky on July 11–12, 2001, to scope the regional Fault Study. Subject-matter experts at this meeting included representatives from the Kentucky Geological Survey, Illinois State Geological Survey (ISGS), Mid-America Earthquake Center, and the University of Memphis. It was mutually agreed that this study be conducted at a site in Massac County, Illinois, at/near Barnes Creek, which is located approximately 11 miles northeast of PGDP (Fig. 1.2). This particular site was selected because of its prominent features, accessibility, and the availability of information form previous studies conducted by ISGS and others. Tasks completed at the Barnes Creek site included visual observations and measurements of features in the banks of Barnes Creek, a ground penetrating radar (GPR) calibration survey, and follow-up GPR survey and direct push technology (DPT) boreholes. This follow-up work was conducted in a potentially faulted area north of the creek.

The purpose of the site-specific Fault Study is to determine if evidence exists of faulting at Site 3A and if the age of any such fault displacement is consistent with results of the regional Fault Study. This study was developed to answer Questions 4 and 5. Site 3A is entirely underlain by the Porters Creek Clay Formation (Fig. 1.2). Initial activities were conducted to determine whether deformation of the Porters

Creek Clay is apparent, based on results of a high-resolution compression wave seismic reflection survey (p-wave survey). Follow-up activities were then conducted to provide higher resolution data, which could better define potential Holocene-age faulting or displacement at relatively shallow depths (from the ground surface to the top of the Porters Creek Clay). These follow-up activities included a horizontal shear wave seismic reflection survey (s-wave survey) and DPT boreholes within a zone that appeared to be faulted based on the p-wave survey at Site 3A.

The purpose of the Geotechnical Study is to determine the variability of the lithology underlying Site 3A and to determine seismic and geotechnical characteristics of the deposits for use in the potential design of an on-site CERCLA waste disposal facility. The Geotechnical Study was structured to support answering Questions 3, 6, and 7. These activities included drilling, sampling, and testing of two deep boreholes, one using a Rotosonic drilling technique and an adjacent boring using a mud rotary drilling technique. The activities also included drilling, sampling, and testing of shallow mud rotary boreholes and seismic cone penetrometer test (SCPT) soundings.

1.3 ORGANIZATION OF REPORT

This Seismic Investigation report is organized into the following chapters:

- Chapter 1, "Introduction," describes the objectives of the Seismic Investigation.
- Chapter 2, "Field Investigation Summary," summarizes the investigation activities and methodologies used in conducting the seismic investigation fieldwork and laboratory testing.
- Chapter 3, "Physical Characteristics," presents the geologic, hydrogeologic, geotechnical, and seismic properties of the Site 3A soil deposits.
- Chapter 4, "Geotechnical Design Model," summarizes use of the collected data to establish a conceptual site model for evaluating the fate and transport of contaminants and also summarizes the evaluation of settlement and bearing capacity for a potential on-site CERCLA waste disposal facility at Site 3A.
- Chapter 5, "Evaluation of Liquefaction," describes the results of the Paleoliquefaction Study and evaluation of liquefaction potential at Site 3A.
- Chapter 6, "Evaluation of Faulting," describes the regional seismic setting, results of the regional Fault Study at the Barnes Creek site, and results of the site-specific Fault Study at Site 3A.
- Chapter 7, "Seismic Design Model," summarizes the evaluation of the PGA and ground motions appropriate for design of a potential on-site CERCLA waste disposal facility at Site 3A.
- Chapter 8, "Summary and Conclusions," summarizes the results of the Seismic Investigation program and presents the conclusions of the study.
- Chapter 9, "References," provides citations for supporting documents used in preparing this Seismic Investigation report and cited in the main text and appendices that follow.

The appendices provide the five technical memoranda that document and present the results (data) from the Seismic Investigation. Included in these technical memoranda are descriptions of the work performed and deviations from the work planned, as well as the seismic reflection profiles, geophysical, DPT, and SCPT logs, and results of the geotechnical testing program. In addition, an appendix is provided that documents detailed calculations made in evaluating the PGA and ground motions for design of a potential on-site CERCLA waste disposal facility at Site 3A.

2. FIELD INVESTIGATION SUMMARY

This chapter summarizes the investigation activities and methodologies used in completing the Seismic Investigation fieldwork and laboratory testing. Chapters 4 through 7 of this report will subsequently present interpretations of the data collected. As shown in Fig. 2.1, the Seismic Investigation was comprised of the following three primary tasks, and this chapter is organized similarly.

- Paleoliquefaction Study

- Fault Study
  - regional Fault Study (Barnes Creek)
  - site-specific Fault Study (Site 3A)

- Geotechnical Study (Acquisition of Seismic and Geotechnical Design Data)
  - regional
  - site-specific

Field investigations were initiated September 28, 2001, and the final drilling operations were completed March 27, 2002. Laboratory analyses were completed during May 2002. The field investigation was conducted in three phases. First, the Paleoliquefaction Study was initiated independently of the Fault Study and the Geotechnical Study. Second, the site-specific Fault Study initial activities were conducted. The project team then met with the EPA and Commonwealth of Kentucky to discuss the results of the initial activities and finalize the plan for conducting the follow-up activities. The third phase consisted of the site-specific Fault Study follow-up activities, the regional Fault Study, and the Geotechnical Study.

Prior to conducting the field activities, existing analytical data were reviewed to determine if Site 3A contained any contamination to address potential concerns with health and safety requirements and waste management concerns. Because the existing data primarily were limited to the northern portion of Site 3A, additional surface soil samples were collected from the southern portions of Site 3A on August 23–24, 2001. The analytical data from this additional sampling indicate that, for the purposes of the Seismic Investigation field program, Site 3A is uncontaminated. Therefore, cuttings from investigation boreholes were spread near the drill sites.

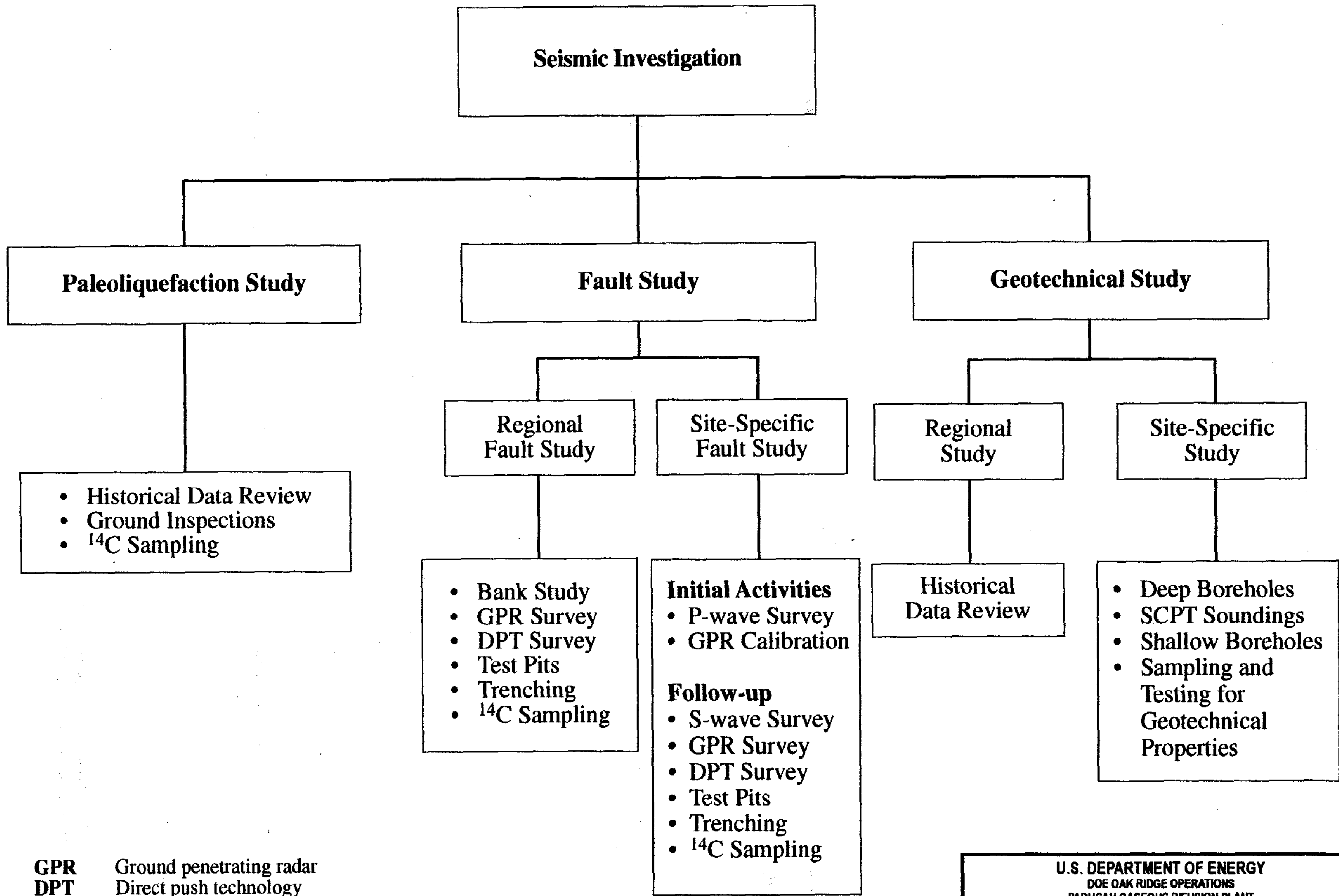
All field investigation activities were conducted by SAIC Engineering, Inc., and its subcontractors. There were no injuries or lost-time accidents associated with this project. The site-specific activities were conducted at/near Site 3A located on the DOE property at the PGDP.

2.1 BACKGROUND ON FIELD INVESTIGATION TECHNIQUES

Several different techniques were used in this Seismic Investigation to gather data and samples for subsequent analysis. The following describes these techniques, highlighting the differences between them and the general type of information that can be obtained.

2.1.1 Intrusive Techniques

Intrusive techniques are methods of exploration that physically penetrate below the ground surface (bgs). The following three intrusive techniques were used for the Seismic Investigation.



GPR Ground penetrating radar
DPT Direct push technology
SCPT Seismic cone penetrometer test

Fig. 2.1. Summary of planned Seismic Investigation activities.

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Figure No. 2.1
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An SPT soil boring is a large-diameter drilled borehole from which discrete samples can be taken. Relatively undisturbed samples can be taken using a 3-inch diameter thin-walled Shelby tube sampler. Disturbed samples can be taken using a 1.5-inch inside diameter split spoon sampler. While taking a split spoon sample, an SPT test can be performed by using a hammer that weighs 140 pounds and falls from a height of 30 inches to progressively drive the sampler into the bottom of the borehole. The SPT provides data on the in-place strength and density of the soil deposit based on the number of blows on the hammer per ft of penetration. The discrete soil samples can be tested in a laboratory for their geotechnical properties.

A DPT borehole is one in which a sampling barrel is hydraulically pushed into the ground, without drilling the borehole. Samples are taken continuously using a 4-ft long by 1.5-inch inside diameter core barrel. The DPT borehole provides a soil core that can be checked for stratigraphy and the presence of organic material suitable for radiocarbon age dating.

An SCPT sounding is a type of probe in which instruments located at the tip of a drill rod are hydraulically pushed into the ground. No samples can be taken, but measurements of tip resistance, sleeve resistance, pore-water pressure, and seismic velocity can be made at frequent depth intervals. The SCPT provides data on the in-place strength, density, and seismic velocity of the soil deposit, as well as the soil stratigraphy.

2.1.2 Non-intrusive Techniques

Non-intrusive techniques are methods of exploration that do not physically penetrate the ground surface. The following two non-intrusive techniques were used for the Seismic Investigation.

A seismic reflection survey is a surface geophysical survey method that uses an energy source to generate seismic waves. These waves propagate through the earth, reflecting off of subsurface features back up to the ground surface. An array of geophones spaced along the ground surface is used to measure the reflected wave. Computer analysis is then used to process and interpret the results. Two types of seismic waves can be generated. Compression, or p-waves, are generated using a vertically induced energy source and propagate through the earth as a series of compressions, similar to sound waves. A p-wave survey is able to penetrate several hundred feet bgs. Shear, or s-waves, are generated using a horizontally-induced energy source and propagate through the earth by microscopically distorting the shape of the soil or rock. An s-wave survey is able to provide higher resolution than a p-wave survey at shallow depths of penetration bgs. Both types of seismic reflection surveys provide data on the subsurface stratigraphy and anomalies that may indicate the presence of faulting.

A GPR survey is a surface geophysical method that uses high-frequency radar antennas to generate electromagnetic waves. These waves propagate through the earth, reflecting off of subsurface features (natural and/or manmade), similar to the seismic reflection waves. The reflected signal is detected at a receiver antenna, and plots of the signals received are used to interpret the results. A GPR survey provides data on distinct shallow anomalies; however, highly conductive soils, such as clays, can attenuate the signal rapidly, which could restrict the depth of penetration to only a few feet.

2.2 PALEOLIQUEFACTION STUDY

The Paleoliquefaction Study was developed to answer Questions 1 and 2 and help in addressing Questions 3, 6, and 7 posed by the Project Core Team.

The planned Paleoliquefaction Study included an historical data review, (regional) ground inspections, and sampling for carbon-14 (^{14}C) age dating of stratigraphic units. The Paleoliquefaction Study activities are summarized in the following paragraphs, and additional details are provided in the corresponding technical memorandum (Appendix A). Results and interpretations of this study are presented in Chap. 5 of this report.

The historical data review phase was conducted by assembling maps, aerial photographs, and other information to identify accessible areas within a 15-mile radius of PGDP that are most likely susceptible to liquefaction. Areas that are susceptible to liquefaction possess a combination of the following characteristics: late Quaternary and Holocene sediments, areas where loose sands are present, and areas where the water table is shallow (i.e., ≤ 15 ft). Geologic maps were used to identify areas of known faulting and Quaternary-age (i.e., Pleistocene and Holocene) deposits, with emphasis on Quaternary sand or gravel deposits. Soil series maps were reviewed to identify areas of sandy or gravelly soils, and aerial photographs were reviewed for evidence of paleoliquefaction features, such as "sand blows." Topographic maps were used to identify drainage features with potentially exposed sections, and to determine accessibility. Water level/elevation maps were compared to topographic maps in an attempt to identify areas with a shallow water table.

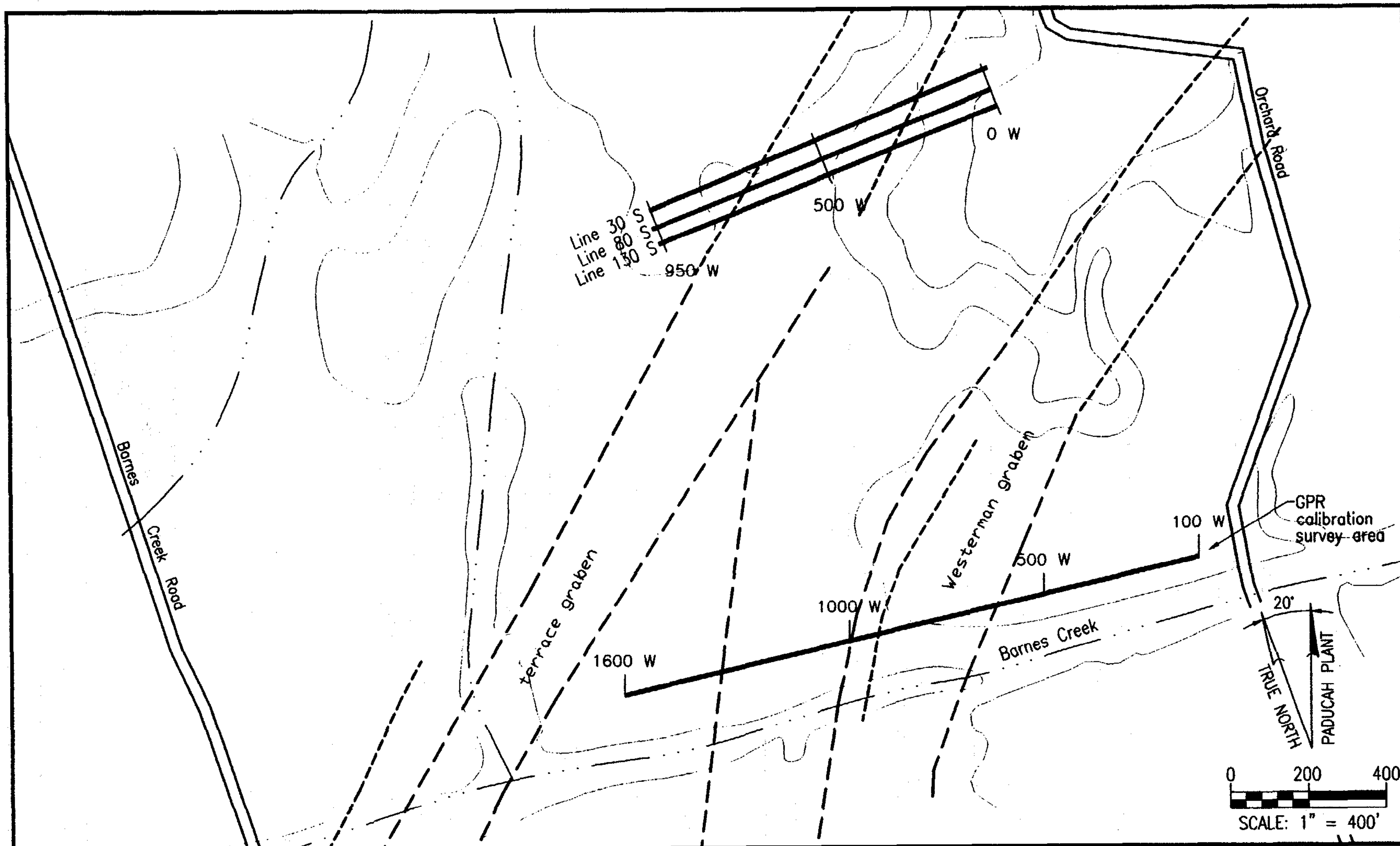
The ground inspection phase consisted of surveying the banks of the Ohio River, visually surveying the banks of Mayfield Creek near highway bridges, and conducting a physical walkdown of two creeks located on the DOE property (Bayou Creek and Little Bayou Creek) and a portion of Barnes Creek. The extent of the ground inspections was limited while site access agreements were being pursued with a large number of landowners. Geologists conducted the Ohio River survey by viewing approximately 100 miles of Ohio River bank from a boat. Although some potential locations for further study were identified, no obvious paleoliquefaction features were observed, even though there was adequate exposure. The geologists then conducted the Mayfield Creek survey at public road crossings and determined that the sediments tended to be clays and silts instead of sands. Vegetation cover prevented full observation of the creek banks. One potential paleoliquefaction feature was observed, but it could not be studied further without an access agreement with the landowner. Fifteen priority sites were identified for further study following the Ohio River and Mayfield Creek surveys (Fig. 2.2). Walkdowns of Bayou Creek and Little Bayou Creek were conducted, but only on those portions that lie on the DOE property. This provided information in the immediate vicinity of Site 3A and PGDP. With the exception of the northern portion of Little Bayou Creek, most of the areas surveyed were vegetated, and the creeks contain few sandy horizons. Two additional potential paleoliquefaction features (one in Bayou Creek and one in Little Bayou Creek) were identified during the walkdowns and warranted further study. Given the initial delays in obtaining access agreements, results of the initial reconnaissance surveys, and preliminary results from work conducted at the Barnes Creek site and Site 3A, a decision was made not to move forward with a detailed study of the priority sites. Therefore, no organic samples were collected for ^{14}C age dating.

2.3 FAULT STUDY

The Fault Study was comprised of two components: a regional Fault Study and a site-specific Fault Study.

2.3.1 Regional Fault Study

The regional Fault Study was developed to support answering Question 4 to determine if faults or postulated faults in the PGDP vicinity have experienced Holocene-age displacement. This study was conducted at the Barnes Creek site located in Massac County, Illinois (Fig. 1.1). Field investigation activities were initiated February 11 and completed February 22, 2002. The regional Fault Study, as originally planned, included a bank study, a GPR survey, a DPT survey, and an excavation of test pits and a trench. The regional Fault Study activities are summarized in the following paragraphs, and additional details are provided in the corresponding technical memorandum (Appendix B). Results and interpretations of the collected data are presented in Chap. 6 of this report.



LEGEND:

- .... Asphalt road
- .... Elevation contour
- .... Inferred fault
- .... Projected fault
- .... GPR survey lines
- .... Stream

Fig. 2.2. Location of Fault Study activities at the Barnes Creek site.

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DATE 07-26-02

A bank study was conducted along an approximately 2600-ft long portion of Barnes Creek, located between Barnes Creek Road and Orchard Road (Fig. 2.3). Visible faulting and other features, including two grabens referred to as the "Westerman graben" and the "terrace graben" (Nelson et al. 1998), were studied at 12 locations. Fourteen organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

A GPR survey was conducted across the suspected projection of the terrace graben, approximately 1100 ft north of Barnes Creek. Three parallel 900-ft lines were surveyed using a 200 MHz antenna. The survey provided high-resolution data of the uppermost sediments and identified areas where the loess was either present or absent, which allowed the locations of the DPTs to be refined.

A DPT survey was conducted across the terrace graben area to potentially identify faulting and/or displacement of relatively shallow units, and collect organic samples for ^{14}C age dating. Ten DPT boreholes were driven across a 450-ft section of the middle line of the GPR survey. The DPT survey produced 403.5 ft of continuous core, which was logged, photographed, and placed in storage. Six organic samples were collected and sent to an off-site laboratory for ^{14}C age dating.

Although excavation of test pits and a trench was planned to collect visible evidence of shallow faulting and additional organic samples for ^{14}C age dating, the DOE investigation team determined that these excavations would not be necessary at the terrace graben area. This decision was based on several factors, but the primary reason the excavations were not performed was the lack of correlation between the surface topography and subsurface faulting in the DPT cores. In addition, the investigation team had collected several organic samples from the bank along Barnes Creek for ^{14}C age dating, and the most likely location of the test pits/trench was in a shallow drainage swale. Lastly, excavations, unlike the other investigation techniques, would have disturbed a significantly larger area of land, possibly causing negative impacts for future use of the property.

2.3.2 Site-Specific Fault Study

The site-specific Fault Study was developed to answer Questions 4 and 5 to determine if evidence exists of faulting at Site 3A and to determine if the age of any such fault displacement is consistent with results of the regional Fault Study. The site-specific Fault Study was conducted in two phases identified as the initial activities and the follow-up activities. First, the initial activities consisted of two nonintrusive geophysical surveys. The results of these two surveys were evaluated and presented to EPA and the Commonwealth of Kentucky. DOE met with EPA and the Commonwealth of Kentucky January 15, 2002, to finalize plans for conducting the follow-up study activities. Then the follow-up activities were conducted. The site-specific Fault Study activities are summarized in the following subsections, and additional details are provided in the corresponding technical memoranda (Appendix C and Appendix D). Results and interpretations of the collected data are presented in Chaps. 3, 4, 5, and 6 of this report.

2.3.2.1 Initial Activities

The initial activities consisted of a high-resolution compression wave seismic reflection survey (p-wave survey) and a GPR calibration survey. These field activities were initiated November 12 and completed December 6, 2001.

The p-wave survey was conducted to determine whether anomalies are present that may suggest the presence of potential young faults at Site 3A. For this study, the term "young fault" was defined as a fault that shows displacement/deformation of the top of the Porters Creek Clay. Four combinations of energy sources [i.e., hammer and cylinder, elastic wave generator (Accelerated Weight Drop), Minivib, and MicroVibrator Source] and receivers were tested along the same 720-ft line at Site 3A. The DOE investigation team then met with subject-matter experts from the Commonwealth of Kentucky on November 15, 2001, to review

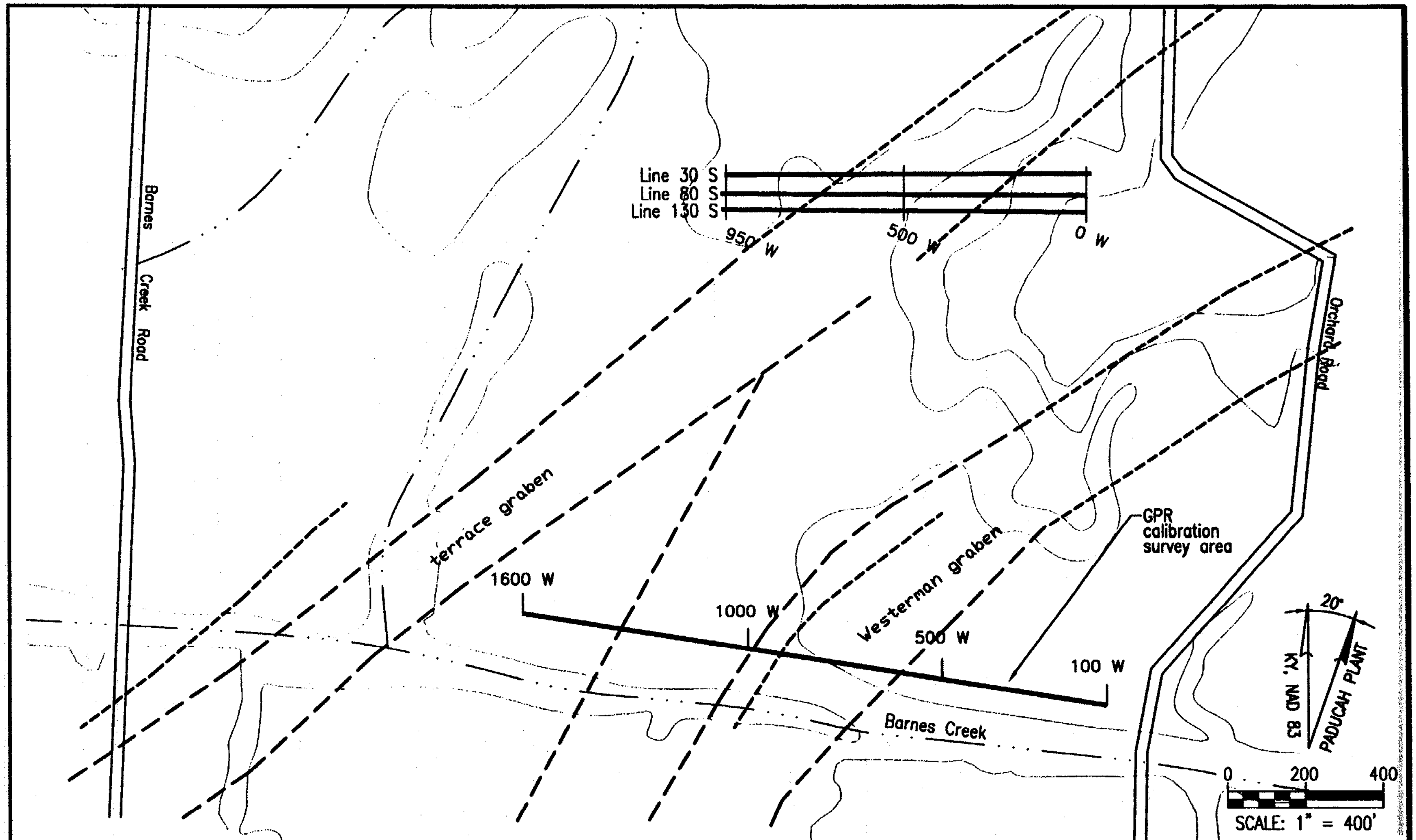

Fig. 2.3. Location of Fault Study activities at the Barnes Creek site.

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the test results and select parameters for conducting the p-wave survey. Approximately 16,000 linear ft of data were then collected along five lines (seven segments) at Site 3A (Fig. 2.4). The p-wave survey used a geophone group interval of 5 ft, shot spacing of 10 ft, 144-channel, 36-fold survey configuration. Several horizons were successfully imaged beneath Site 3A, including the top of Mississippian-aged limestone bedrock, Cretaceous-aged McNairy Formation (lower sand facies), and portions of the Paleocene-aged Porters Creek Clay. The results also indicated potential young faults extending from the limestone bedrock up into the Porters Creek Clay; therefore, a follow-up horizontal shear wave seismic reflection survey (s-wave survey) was recommended at Site 3A.

The GPR calibration survey was conducted to determine whether GPR was capable of penetrating local clays and silts to identify subsurface features. This calibration survey was performed at the Barnes Creek site in Massac County, Illinois. A 1500-ft test line was established approximately 50 ft north of Barnes Creek. Four GPR surveys/tests were conducted along this line using 200, 100, 80, and 16 MHz antennas. A blind test was conducted (i.e., the locations of faults and other features that are visible in Barnes Creek were not provided to the experts conducting the GPR tests until the data had been interpreted). After testing the GPR equipment at the Barnes Creek site, the testing was continued along a 750-ft-long test line at Site 3A using 200 and 40 MHz antennas (Fig. 2.4). The survey at Site 3A indicated that neither high- nor low-frequency GPR would provide suitable resolution of the geology at the site; as a result, no follow-up GPR survey was recommended for Site 3A. The survey also indicated that high frequency GPR (i.e., 200 MHz) would provide the greatest resolution of the geology at the Barnes Creek site; therefore, a follow-up GPR survey was recommended for the Barnes Creek site.

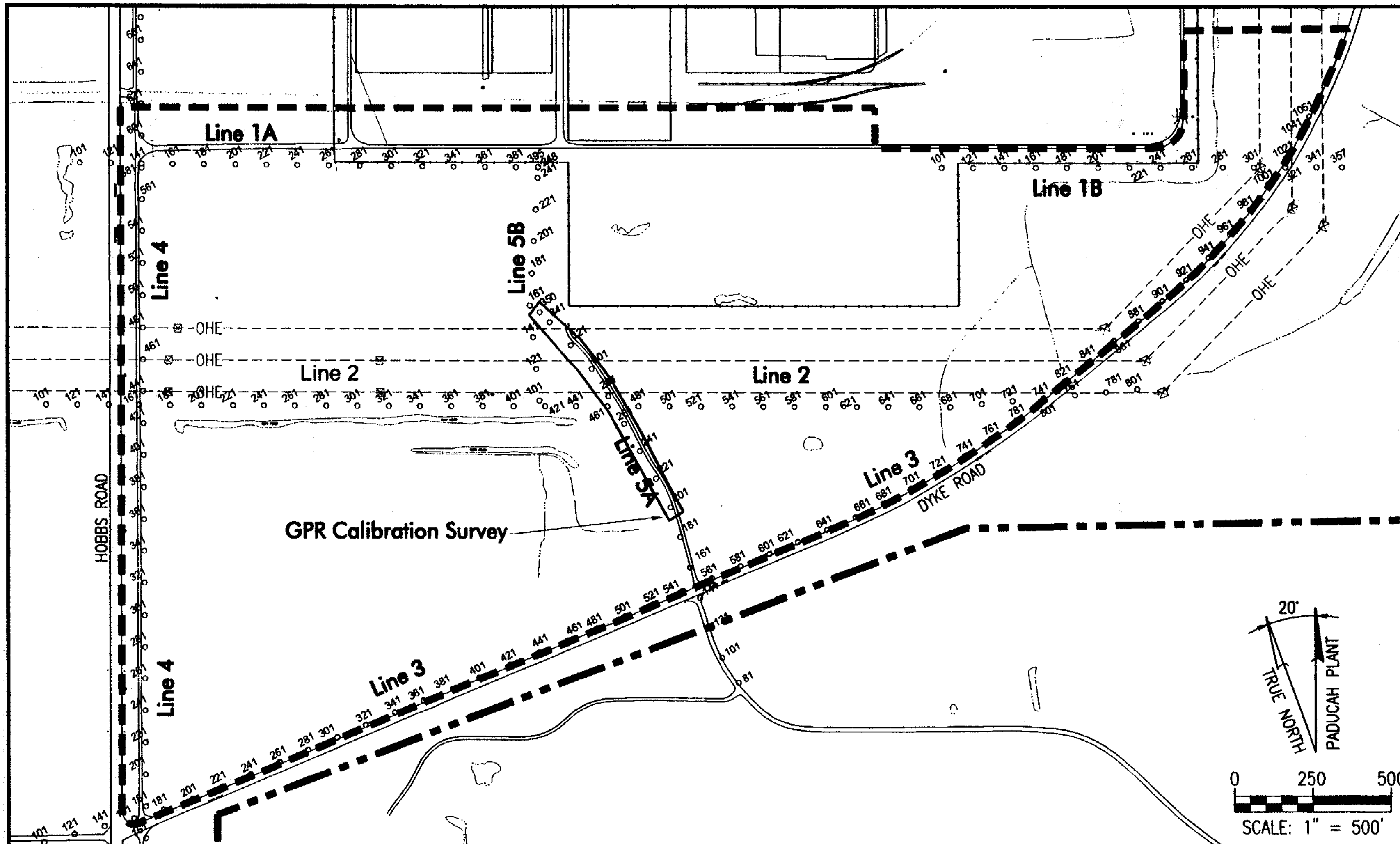
2.3.2.2 Follow-up Activities

The planned follow-up activities at Site 3A consisted of an s-wave survey, a GPR survey, a DPT survey, and excavation of test pits and a trench. After meeting with the regulatory agencies on January 15, 2002, the follow-up field activities were initiated January 30 and completed March 8, 2002.

The s-wave survey was conducted to determine whether anomalies are present that may suggest the presence of potential shallow faults at Site 3A. When DOE met with EPA and the Commonwealth of Kentucky on January 15, 2002, the anomalies observed from the p-wave survey results were used to identify the locations for the s-wave lines. Approximately 2300 linear ft of data were then collected along two lines at Site 3A (Fig. 2.5). The s-wave survey used a geophone group interval of 2 ft, shot spacing of 2 ft, 96-channel, 48-fold configuration. Several shallow horizons were successfully imaged beneath Site 3A, including the loess, a firm sand unit underlying the loess, and the Porters Creek Clay. Several faults extending up to or near the bottom of the loess unit (approximately 20 ft bgs) were identified. The s-wave survey results support the general conclusions derived from the previous p-wave survey.

The GPR survey was originally planned to determine whether anomalies are present that may suggest the presence of potential faults in shallow sediments at Site 3A, and to refine the locations of the planned intrusive activities. The results of the previous GPR calibration survey indicated, however, that the GPR technology is incapable of penetrating clays and silts at Site 3A with sufficient resolution to identify subsurface features. As a result, DOE, EPA, and the Commonwealth of Kentucky mutually agreed that the GPR survey should not be conducted as one of the follow-up activities at Site 3A.

The purpose of the DPT survey was to collect soil cores to identify potential Holocene-age faults and displacement at relatively shallow depths. Ten DPT boreholes were driven along the two s-wave lines to depths ranging from 21 ft to 40 ft; in addition, an eleventh DPT borehole was driven at one of the planned shallow boring locations (SB-04). The DPT survey at Site 3A produced nearly 400 ft of continuous core, which was logged, photographed, and placed in storage. The soil cores allowed the stratigraphy to be observed. Faulting was observed at a depth of 20 to 28 ft in a DPT borehole located near the southern



LEGEND:

- Asphalt road
- Gravel road
- PGDP boundary
- Site 3A boundary
- Railroad tracks
- Fence line
- Stream
- OHE Electrical utility lines
- O501 P–wave survey location

Fig. 2.4. Location of Fault Study initial activities at Site 3A.

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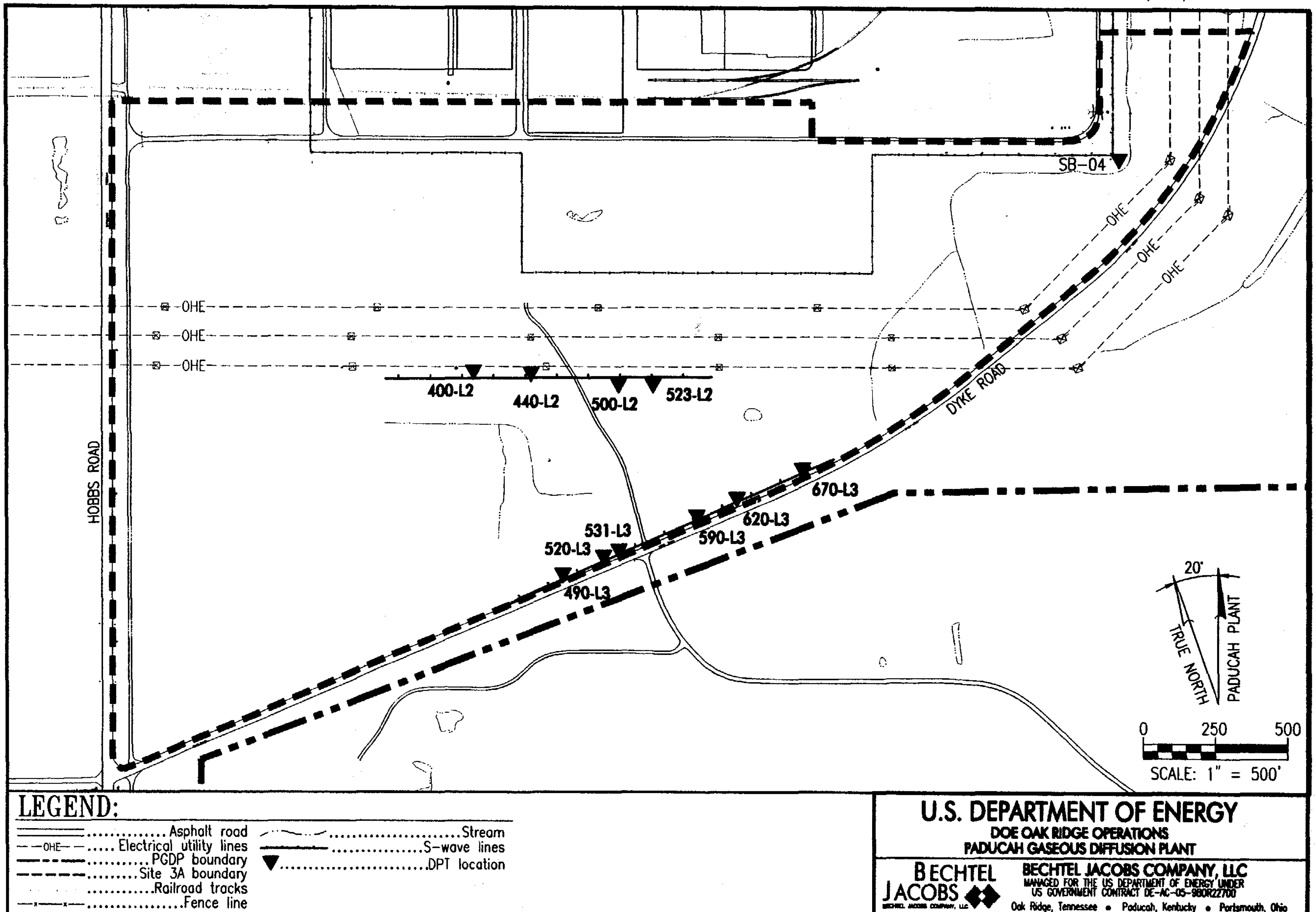

Fig. 2.5. Location of Fault Study follow-up activities at Site 3A.

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Figure No. /99049/DWGS/P34SK512

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boundary of Site 3A (Line 3, Station 531). Five organic samples also were collected and sent to an off-site laboratory for ^{14}C age dating. Each DPT borehole was plugged and abandoned in accordance with applicable regulations.

Although excavation of test pits and a trench were originally planned to collect visible evidence of shallow faulting, the DOE investigation team determined that these excavations should not be performed at Site 3A, based on the results of the DPT survey and the site-specific Geotechnical Study. This decision also was based on field conditions that would prohibit these activities (e.g., high water levels, excessive excavation required to reach required depths, and obstructions, including trees/woods, paved roads, an underground utility, and potential wetlands). The DOE investigation team has suggested that if additional data are required, a safer approach may include installing closely-spaced DPT boreholes in lieu of the test pits and trench.

2.4 GEOTECHNICAL STUDY

The Geotechnical Study was developed to acquire seismic and geotechnical design data that would support answering Questions 3, 6, and 7; namely, to determine if there is potential for future liquefaction at Site 3A and to determine the PGA and the characteristics of ground motion to use in the design of a potential on-site CERCLA waste disposal facility. The Geotechnical Study consisted of regional and site-specific investigations. The regional investigation consisted of obtaining time histories and other pertinent backup information used by Risk Engineering, Inc. (REI) (1993 and 1999) to determine the bedrock characteristics at the PGDP soil/rock interface, the soil column, and the PGA, as developed in previous PGDP studies. The regional investigation did not include any field work. The results of the required investigation are summarized in (BJC 2002c) and discussed further in Chap. 7 of this report.

The site-specific Geotechnical Study included the installation of seismic cone penetrometer test (SCPT soundings), deep borings, shallow borings, and sampling/testing for seismic and geotechnical properties at Site 3A. The acquisition of basic seismic geotechnical information that would be required for future design of a potential on-site CERCLA waste disposal facility is included in this task to save time and money on any future site characterization. Field investigation activities were initiated February 12 and completed March 26, 2002. The Geotechnical Study activities are summarized in the following paragraphs, and additional details are provided in the corresponding technical memorandum (Appendix E). Results and interpretations of the collected data are presented in Chaps. 3, 4, and 5 of this report.

Fourteen SCPT soundings were conducted at 11 locations at Site 3A to support the future design of a potential on-site CERCLA waste disposal facility (Fig. 2.6). The upper six ft of each SCPT location were augered by hand, and the total depths of the SCPT soundings ranged from approximately 10 ft (refusal) to 70 ft bgs. A total of 623.4 ft were sounded. Continuous tip, sleeve, and pore pressure measurements were collected from 6 ft bgs to total depth in each SCPT sounding. Twenty-nine pore pressure dissipation tests were conducted in varying lithologies with different permeabilities to assess the potential of the sediments to liquefy and assess the competency of the clays. Shear-wave velocities were measured at approximate 3-ft intervals in each SCPT sounding, beginning at a depth of 6 ft bgs. No samples were collected and no drill cuttings were generated. Each SCPT sounding was plugged and abandoned in accordance with applicable regulations.

Two deep boreholes were drilled at Site 3A (Fig. 2.6). The first deep borehole (DB-01) was drilled to a total depth of 359 ft using a Rotosonic drilling technique. The continuous core was collected, logged, and placed in storage. A downhole geophysical survey (natural gamma log) of the entire borehole was conducted. The core and gamma log were evaluated to select specific depths for collecting soil samples in the second borehole. The second, adjacent, deep borehole (DB-02) was drilled to bedrock (total depth 400.3 ft)

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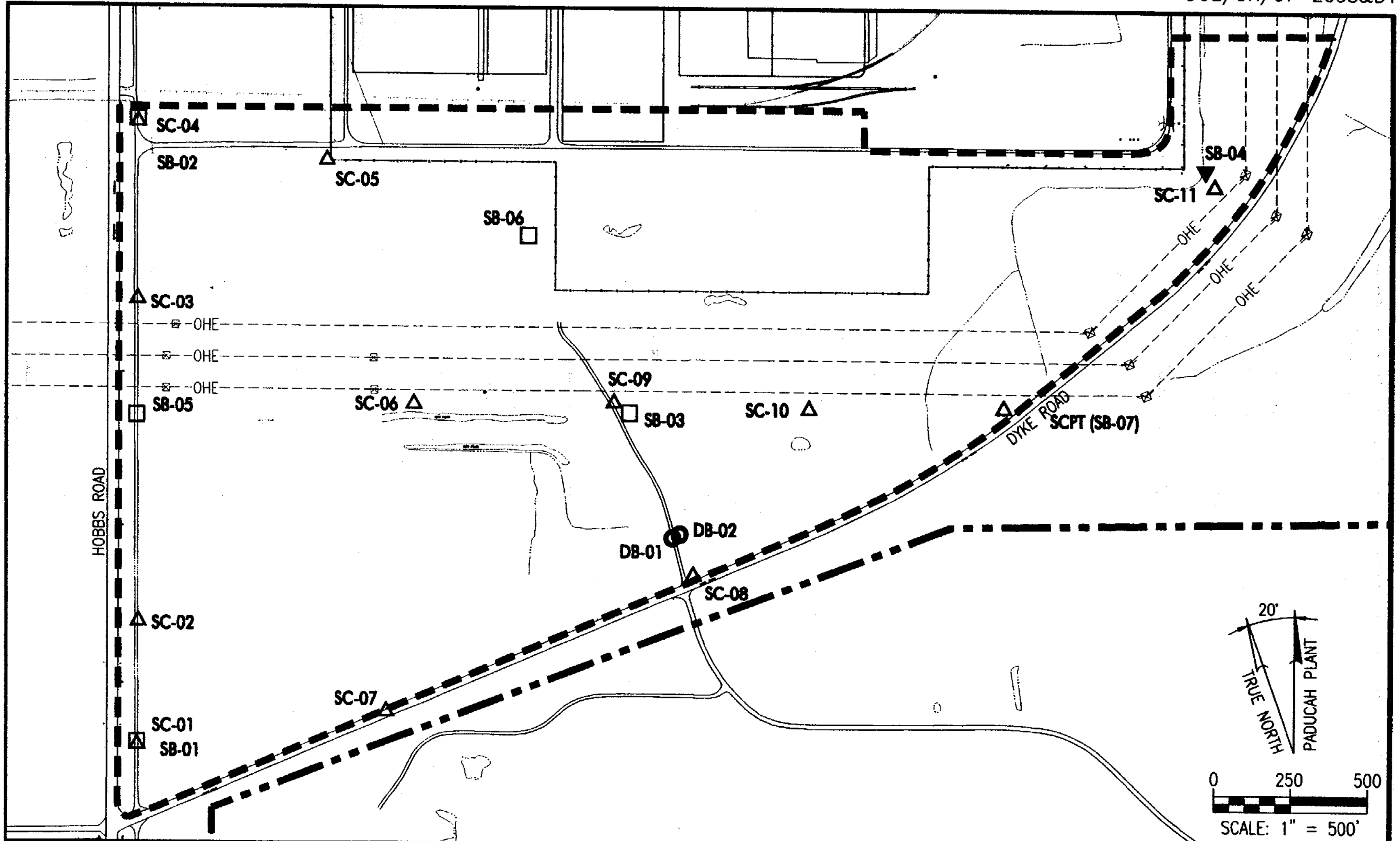

LEGEND:

............Asphalt road	......................Stream
—OHE— ..... Electrical utility lines	..........................DPT location
...........PGDP boundary	..............Deep boring location
.........Site 3A boundary	............Shallow boring location
...........Railroad tracks	.....................SCPT location
................Fence line	

Fig. 2.6. Location of Geotechnical Study activities at Site 3A.

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using a mud rotary technique, which was necessary to collect the planned samples and conduct the planned logging. Standard Penetration Test (SPT) sampling was conducted continuously to a depth of 75 ft and at approximately 20-ft intervals thereafter to a depth of 186 ft. Several samples were collected from the second deep borehole. Eleven Shelby tube samples were collected at depths ranging from 4 ft to 134 ft bgs and sent to a laboratory for measurement of physical soil properties. Thirty-two split spoon samples were collected at depths ranging from the ground surface to 186 ft bgs, and 12 of these samples were sent to a laboratory for measurement of geotechnical index properties. A downhole seismic survey (shear wave velocity log) of the borehole was conducted to determine shear wave velocities for use in liquefaction analyses, deformation studies, and development of a site-specific PGA value. Both deep boreholes were plugged and abandoned in accordance with applicable regulations. Because Site 3A is not contaminated, cuttings from the mud rotary borehole were spread near the drill site.

Five shallow boreholes were drilled at Site 3A using a mud rotary technique to support the future design of a potential on-site CERCLA waste disposal facility (Fig. 2.6). The total depths of the borings ranged from 52 ft to 70 ft bgs. Two additional shallow boreholes were originally planned, but heavy rainfall created accessibility concerns for the truck-mounted rigs. Because the DPT/SCPT rig was mounted on a tracked vehicle, the DOE investigation team converted the planned shallow borings into one DPT borehole (SB-04) and one SCPT sounding (SB-07). The DPT borehole (SB-04) is documented with the site-specific Fault Study activities. SPTs were conducted continuously throughout the depth of the shallow boreholes in accordance with American Society for Testing and Materials (ASTM) D1586. Several samples were collected from the shallow boreholes. Thirty-three Shelby tube samples were collected at depths ranging from 4 ft to 67 ft bgs, and 29 of these samples were sent to a laboratory for measurement of physical soil properties. In addition, 121 split spoon samples were collected at depths ranging from the ground surface to 70 ft bgs, and 36 of these samples were sent to a laboratory for measurement of geotechnical index properties. Three organic samples also were collected from two shallow boreholes and sent to an off-site laboratory for ^{14}C age dating. All shallow boreholes were plugged and abandoned in accordance with applicable regulations. Since Site 3A is not contaminated, cuttings from the mud rotary boreholes were spread near the drill sites.

Numerous Shelby tube and split-spoon samples that were collected from the deep and shallow borings at Site 3A. Relatively undisturbed soil samples were excavated from the Shelby tubes and analyzed for physical soil properties (e.g., in-place density, vertical permeability, triaxial compressive strength, and one-dimensional consolidation). Disturbed soil samples were collected from the split spoon samples and analyzed for geotechnical index properties (e.g., specific gravity, grain size, Atterberg limits, and moisture content) and contaminant transport properties (e.g., contaminant partitioning coefficients for ^{99}Tc and ^{239}Np). ASTM standards were used to collect and analyze the samples.

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3. PHYSICAL CHARACTERISTICS

This chapter describes the physical characteristics of the subsurface at PGDP, including site conditions relative to geography and physiography, seismology, geology, and hydrology. Results of the field characterization at Site 3A are presented, including site-specific stratigraphy, water levels, and geotechnical and seismic soil properties. These results are subsequently used in Chaps. 4 through 7 of this report to evaluate the geotechnical design model, liquefaction potential, faulting potential, and seismic design model.

3.1 GEOGRAPHY AND PHYSIOGRAPHY

PGDP is located in western McCracken County, Kentucky, approximately 3 miles south of the Ohio River and approximately 10 miles west of the city of Paducah (see Fig. 1.1). PGDP is located in the Jackson Purchase Region of western Kentucky, at the northern tip of the Mississippi Embayment portion of the Atlantic Coastal Plain physiographic province. The Mississippi Embayment is a large sedimentary trough oriented north-south, which received sediments from the middle of the North American continent. The area is bounded on the north and east by the Illinois Basin, an area of low plateaus on stratified sedimentary rock within the Highland Rim portion of the Interior Low Plateau physiographic province.

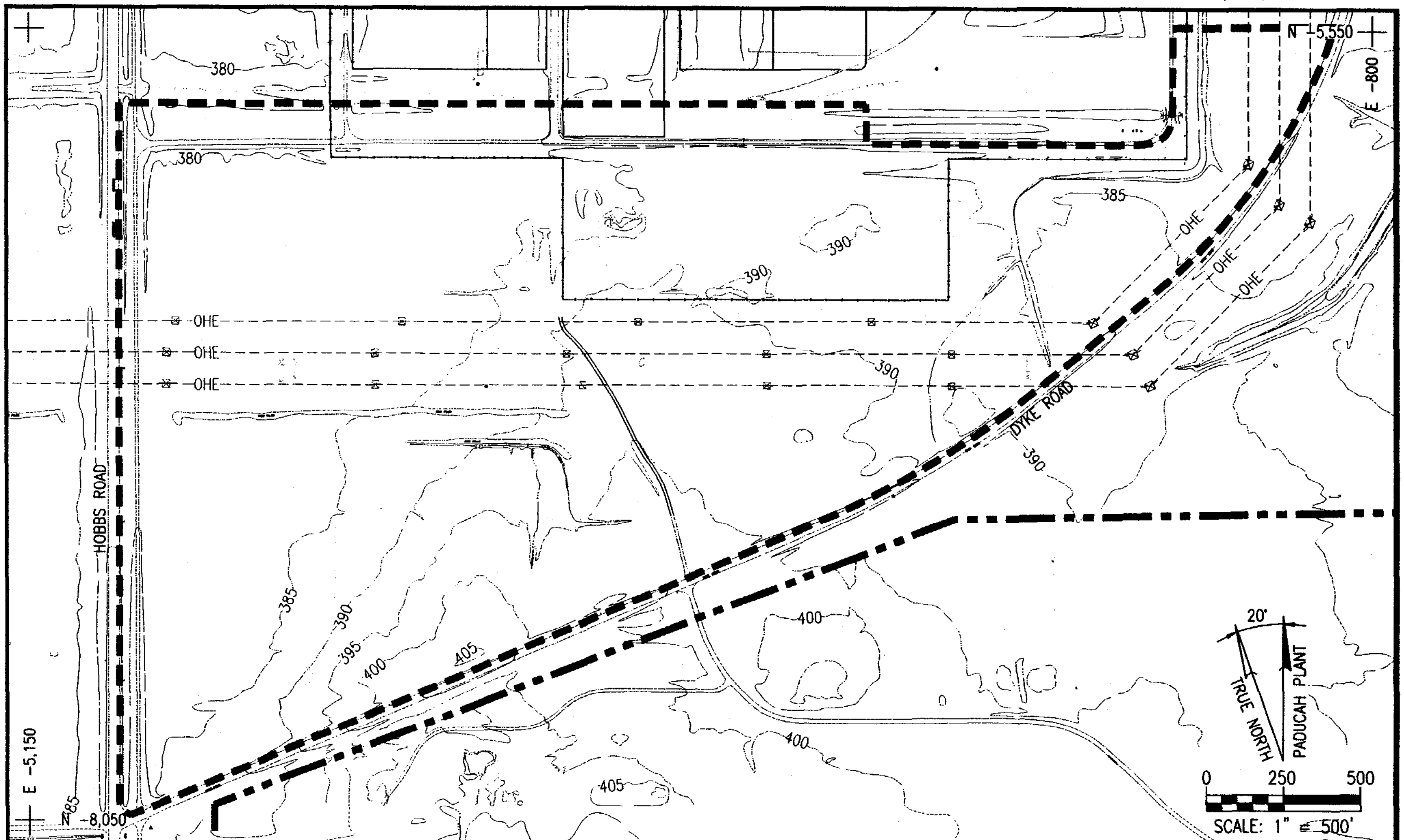
PGDP is situated in an area characterized by low relief. Elevations vary from approximately 350 to 405 ft mean sea level (msl) across the DOE property, with the ground surface sloping at an approximate rate of 27 ft/mile toward the Ohio River. Two main topographic features dominate the landscape in the area: a loess-covered plain at an average elevation of 390 ft msl and the Ohio River floodplain zone, dominated by alluvial sediments, at an average elevation of 315 ft msl. The terrain is modified slightly by the drainage systems associated with the two principal streams in the area: Bayou Creek and Little Bayou Creek. These northerly flowing streams have eroded small valleys that are approximately 20 ft below the adjacent plain.

Site 3A is located on the loess-covered plain. The topography at Site 3A is relatively flat, varying in elevation from a high of approximately 405 ft msl in the southern portion of the site alongside Dyke Road to a low of approximately 380 ft msl in the northeastern and northwestern corners of the site (Fig. 3.1). Drainage is predominantly by overland flow to several man-made ditches located northeast, northwest, and southwest of the site. These ditches ultimately discharge to Little Bayou Creek to the east and Bayou Creek to the west.

3.2 REGIONAL SEISMOLOGICAL SETTING

Several large-scale fault systems in Paleozoic and younger rocks have controlled much of the region's seismic history. To the southwest and northeast of PGDP is a dogleg-shaped, failed rift composed of two segments, the northeast-trending Reelfoot rift of the Mississippi Embayment and the east-trending Rough Creek graben of the Illinois Basin (Fig. 3.2).

Seismic activity in the PGDP area is primarily the result of continental compression that is reactivating the New Madrid seismic zone (NMSZ), centered 62 miles to the southwest of PGDP (Fig. 3.2). The NMSZ is a zone of dense microseismic activity associated with the Reelfoot rift. The Reelfoot rift system subsided rapidly throughout Paleozoic time [225 to 570 million years ago, serving as a center of sedimentation for the Illinois Basin. Beginning in Cretaceous time (65 to 135 million years ago), the Reelfoot rift area again subsided, forming the Mississippi Embayment. Consequently, northeast-trending faults were rejuvenated in and near the NMSZ. Three major earthquakes occurred in the NMSZ in late 1811 and early 1812, each of which was estimated to have a magnitude greater than 7.



LEGEND:

....	Asphalt road
....	Gravel road
....	PGDP boundary
....	Site 3A boundary
....	Fence line
....	Railroad tracks
....	Stream
....	Contour (5' interval)
----OHE-	Electrical utility lines

Fig. 3.1. Ground surface elevations at Site 3A.

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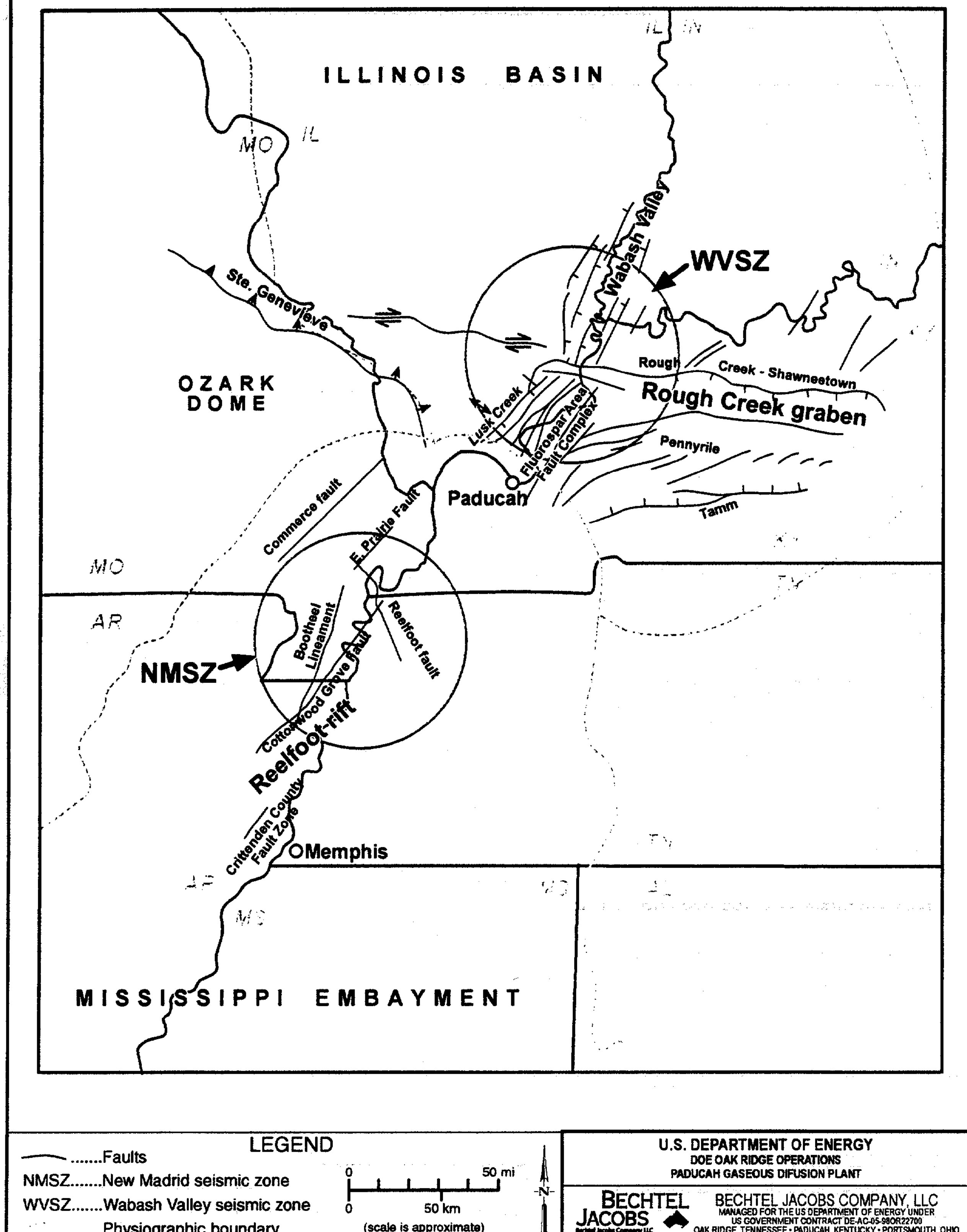

Fig. 3.2. Regional tectonic map.

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To the north of PGDP in the Illinois Basin is the Wabash Valley seismic zone (WVSZ), presently an area of diffuse seismicity. Principal structures within the WVSZ are the Rough Creek graben, Hicks Dome/Fluorspar Area fault complex, and the Wabash Valley fault system. Post-Pennsylvanian displacements along the Rough Creek-Shawneetown fault system split the Illinois Basin into two unequal parts: a broad, but shallow, basin to the north and a narrow, but deep, basin to the south. Several researchers have found geologic evidence to demonstrate that the WVSZ has a history of earthquakes of magnitudes greater than 6 (REI 1999).

3.2.1 PGDP Seismological Setting

PGDP is located near the inferred junction of the Reelfoot rift, which contains all the active NMSZ faults, and the WVSZ Rough Creek graben, which has a lower rate of seismic activity. The boundary between these two structures is not well defined.

Geologic maps of the PGDP area delineate few faults. The closest mapped faults in Kentucky (Fig. 3.3) are located approximately 4 miles east and 5 miles northwest of PGDP (Olive 1980). Recent mapping in the southern Illinois Fluorspar Area fault complex provides convincing evidence of widespread tectonic faulting of Cretaceous and younger units (less than 135 million years ago). The style and trend of these faults are inconsistent with the contemporary stress regime and with the inferred style and trend of active faults in the New Madrid area (Nelson et al. 1997). Traces of several faults in southern Illinois trend toward PGDP.

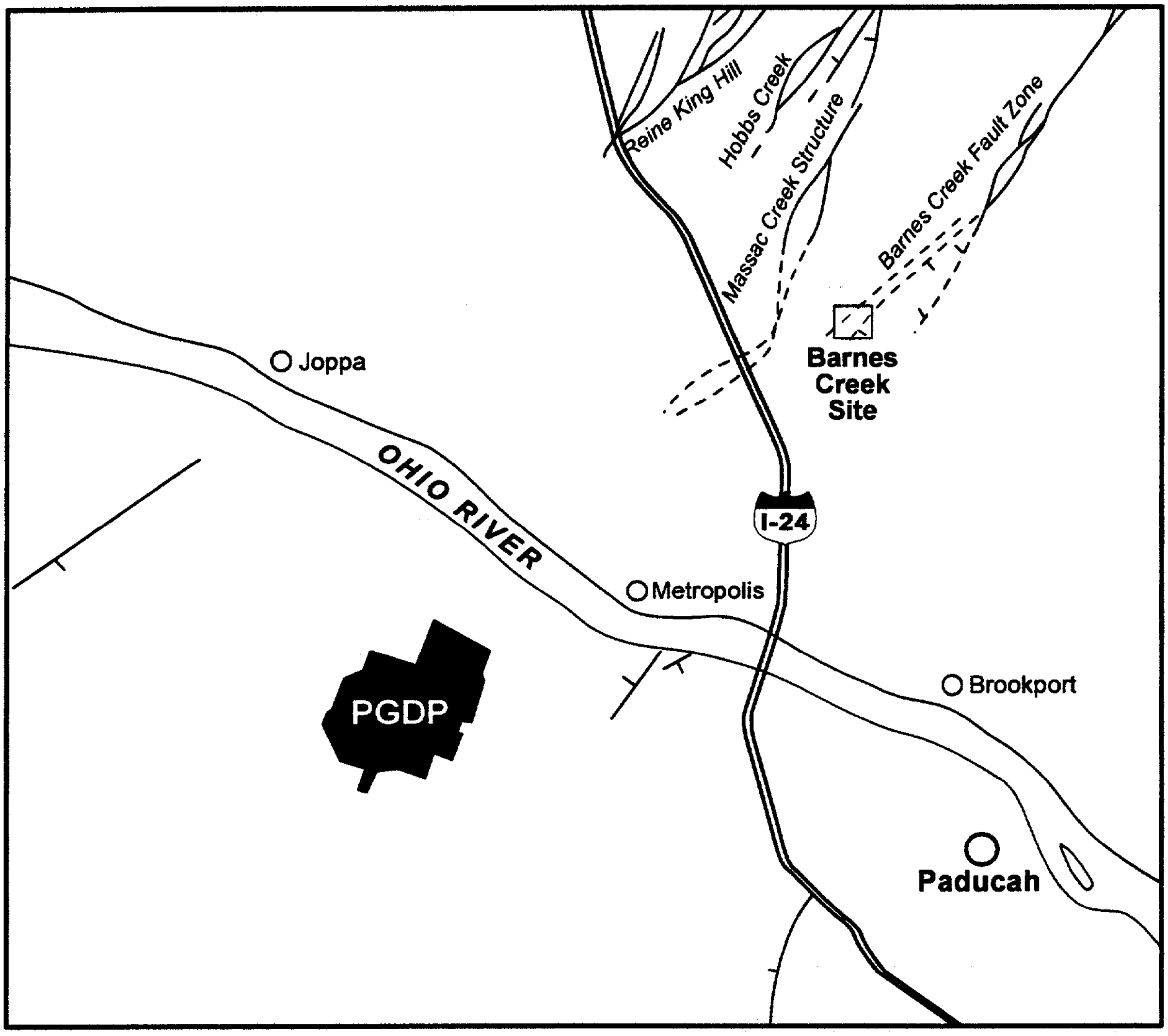
The Barnes Creek fault zone, if extended below the Mississippi Embayment, would be the most likely fault to pass through or near PGDP. Where exposed in southern Illinois, the Barnes Creek fault zone is a single fault or a zone of sub-parallel faulting less than 0.25 miles wide. The vertical separation along the fault typically is less than 100 ft. Nelson et al. characterizes the latest displacement along the fault zone as probably early Pleistocene (older than 13,000 to 14,000 years ago) (Nelson et al. 1996). Undeformed Holocene (younger than 10,000 years ago) gravels and silts overlie splays of the fault. This fault zone was investigated further during the regional Fault Study, as discussed in Chap. 6 of this report.

The other fault zone likely to pass below or near PGDP (probably on the west side) is the Massac Creek structure of the Hobbs Creek fault zone. Nelson et al. (1996) interprets this graben in the Hobbs Creek fault zone to have formed in Miocene to early Pleistocene time (1.6 to 24 million years ago).

The Kentucky Geological Survey has used several techniques to define the geologic structure of western Kentucky, including the PGDP area. Based on imagery from side-looking airborne radar, Drahovzal and Hendricks postulate two regional northeast-southwest lineaments extending through PGDP, as well as three other nearby lineaments within or adjacent to DOE property (Fig. 3.4) (Drahovzal and Hendricks 1996). These lineaments closely correspond to regional lineaments in the top-of-basement map for the Paducah area, which may be related to faulting.

Street and Langston present the interpretation of seismic geophysical surveys within the DOE property (Street and Langston 1998). The seismic survey data were collected along six main transects located east and north of the main plant (Fig. 3.4). In the report, researchers identify the following three anomalies that they attribute to faulting:

- A structural depression on seismic wave reflectors north-northwest of PGDP suggests the presence of a large graben, progressing upward from bedrock into the Lower Continental Deposits (Pleistocene age: 11,000 to 1.6 million years ago) that trends northeast-southwest. The geophysical survey profiles suggest that the graben measures approximately 0.6-miles wide in the Lower Continental Deposits.



LEGEND

Fault lines

0 3 mi
0 2 km
(scale is approximate)

N

Fig. 3.3. Closest mapped faults to PGDP.

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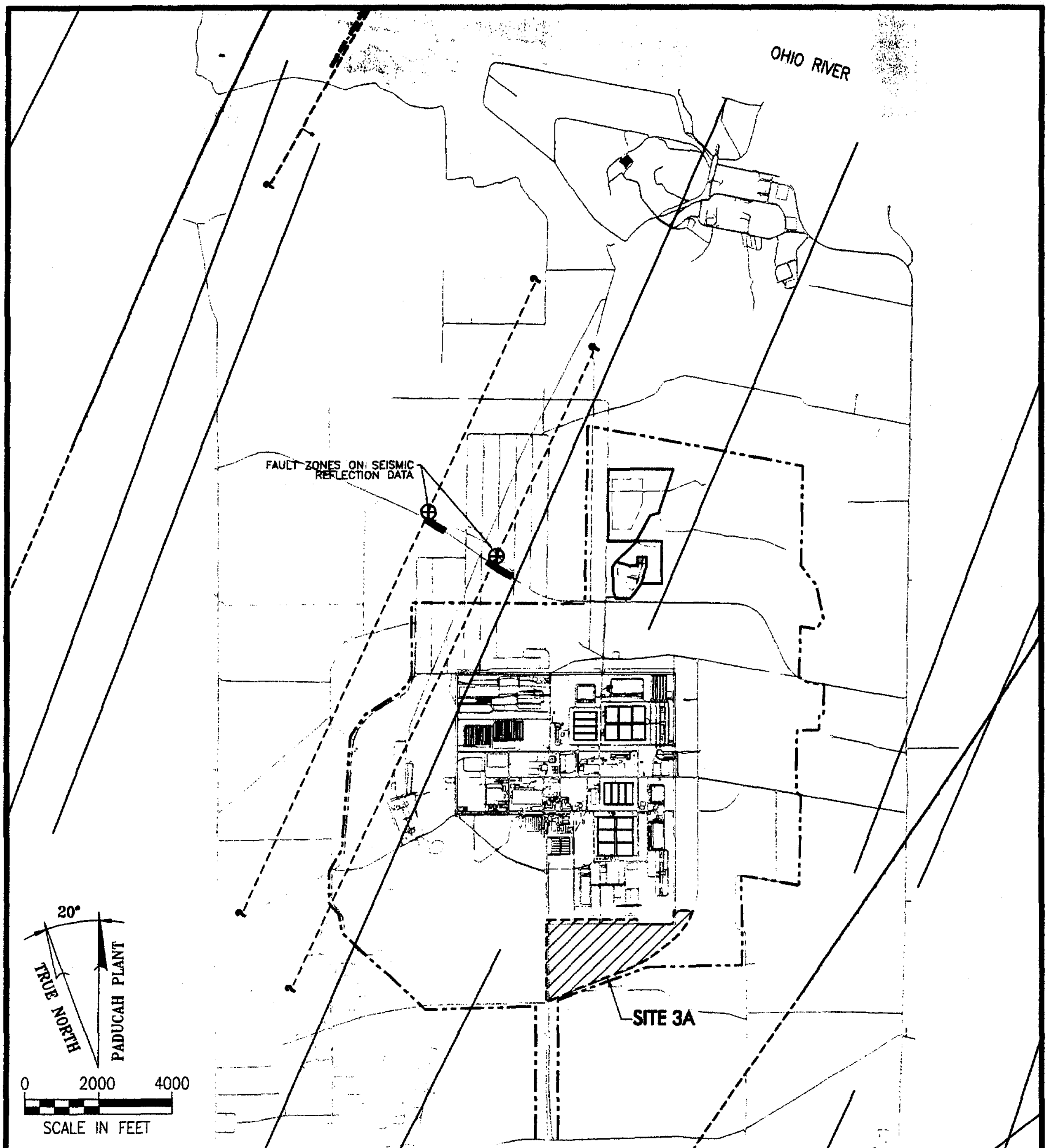

LEGEND:

- ....DOE boundary
- .....Lineament feature from Drahovzal & Hendricks (1996)
- ... Potential fault as defined by Street & Langston (1998)

Fig. 3.4. Postulated faults and lineaments in the PGDP vicinity.

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- Three geophysical survey lines on the north side of the plant-secured area indicate a northeast-southwest trending zone of displacement in the base of the Lower Continental Deposits, with the down-thrown block to the northeast.

- Two geophysical survey lines on the east side of the plant indicate a northeast-southwest trending structure. This anomaly most likely represents a buried erosional feature, the south bank of the ancestral Tennessee River, which flowed through the area during Pleistocene time (11,000 to 1.6 million years ago).

3.2.2 Regional Seismic Activity

Recent seismic events have occurred in both the NMSZ and WVSZ. Figure 3.5 shows the distribution and magnitude of seismic events in the PGDP region for the period 1800 through 2002 (USGS 2002a).

3.2.2.1 New Madrid Seismic Zone

The principal seismic activity within the NMSZ is interior to the Reelfoot rift. Earthquakes are occurring at depths between 2 and 7 miles bgs in Precambrian granites and Lower Paleozoic sedimentary rocks.

The NMSZ is the principal area to consider in modeling future seismic events that might impact PGDP. The principal active faults of the NMSZ are the Blytheville arch (including a northeastern extension as the Cottonwood Grove fault), the Bootheel lineament, the Reelfoot fault (including its northern extension to northwest of New Madrid and southern extension to north of Dyersburg), and, with lower confidence, the East Prairie fault (Fig. 3.2).

Researchers of the NMSZ have associated the following three 1811–1812 New Madrid earthquakes to a specific fault by using historical accounts and geological evidence. The 1999 PGDP seismic hazard analysis used those faults, augmented to the north, to characterize the main pattern of seismicity in the NMSZ (REI 1999).

- The December 11, 1811, NMSZ event (magnitude 8.1) occurred in northeast Arkansas and is associated with a strike-slip rupture on the Blytheville arch-Cottonwood Grove fault, or with the Blytheville arch-Bootheel lineament. The 1999 PGDP seismic hazard analysis applies the former interpretation, with a fault length of 75 miles.

- The January 23, 1812, NMSZ event (magnitude 7.8) occurred in southeast Missouri and is associated with a strike-slip rupture on the East Prairie fault. The possibility that the East Prairie fault connects with the Fluorspar Area fault complex is represented by an East Prairie extension.

- The February 7, 1812, NMSZ event (magnitude 8.0) also occurred in southeast Missouri and is associated with a thrust rupture on the Reelfoot fault. The 1999 PGDP seismic hazard study uses a length of 43 miles for the Reelfoot fault.

  Of the faults associated with the three great earthquakes of 1811 and 1812, the only known surface fault is the Reelfoot reverse-fault scarp. Apparently, the first and second events occurred on strike-slip faults, thereby leaving no scarps. Investigators have interpreted the area earthquake chronology prior to recorded history from scarp-derived colluvium and graben-fill sediments (REI 1999). Three faulting events are thought to have occurred along the Reelfoot rift within the last 2400 years as follows: between 1000 and 1220 years ago, between 350 and 740 years ago, and the 1811-1812 events. Thus, the recurrence interval of the Reelfoot fault is estimated to be from 400 to 500 years.

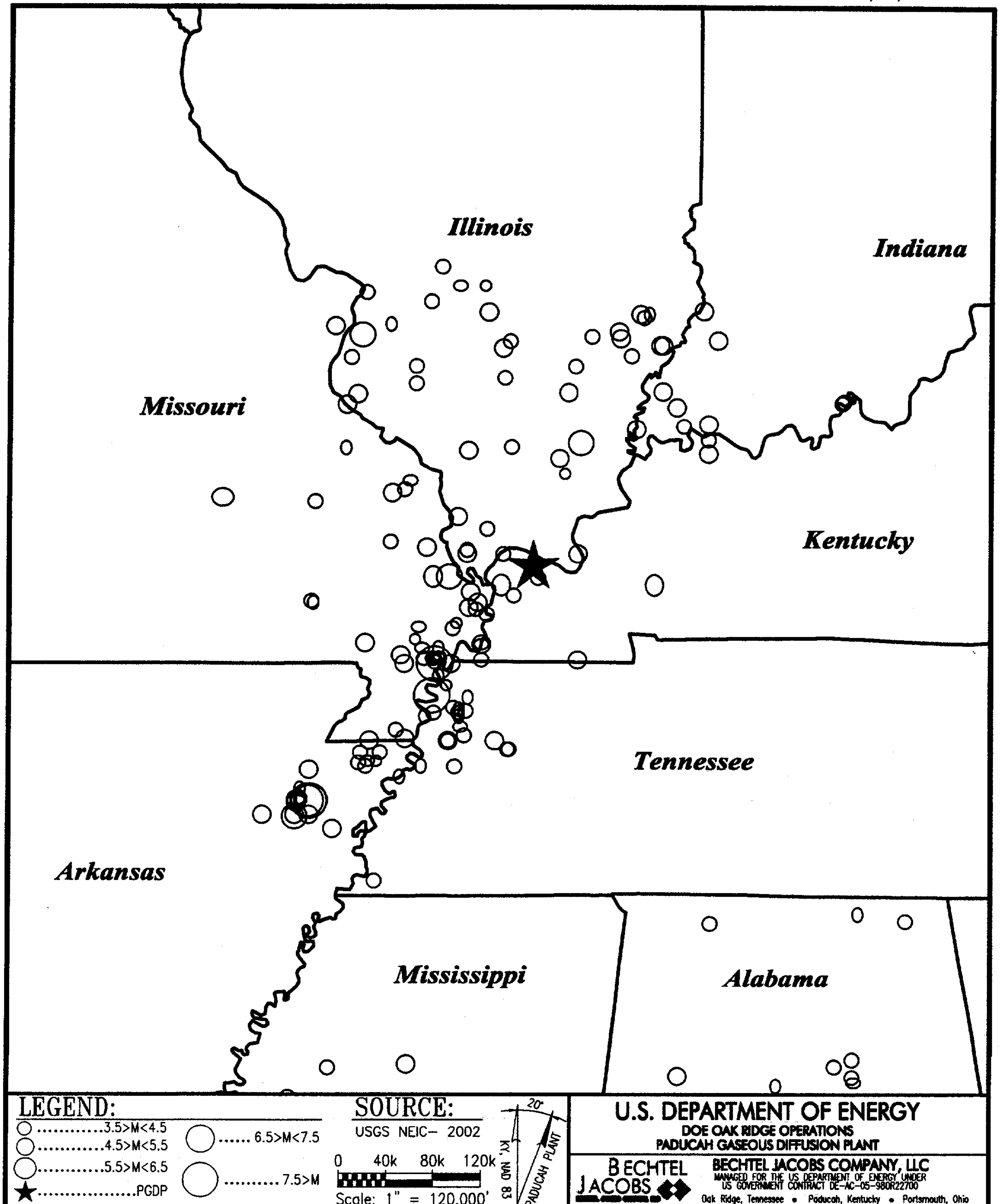

Fig. 3.5. Seismic activity within the New Madrid and Wabash Valley seismic zones.

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There are only three known faults in the study region (other than the principal NMSZ faults) for which offset of Holocene (less than 10,000 to 12,000 years ago) sediments has been documented (Fig. 3.2). Two of these, the Commerce/Benton Hills and the Crittenden County fault zone lie within or near the northwest and southeast margin zones, respectively, of the Reelfoot rift. These are most probably rift-bounding normal faults that have been reactivated as thrust faults or strike-slip faults in the current east-west compressional stress regime. With lengths of more than 12 to 19 miles, these faults could generate maximum earthquakes of magnitude 7.

Paleoliquefaction studies of sand blow deposits and sand dikes are a particularly powerful tool in deciphering the paleoseismology of the NMSZ (REI 1999). Paleoliquefaction studies in the Western Lowlands of Missouri have identified four paleoseismic events dating from 17,000 to 23,000 years ago, 9000 to 13,500 years ago, 980 to 1760 years ago, and 460 to 560 years ago. Researchers speculate that the seismic source of the Western Lowland earthquakes may have been the Commerce fault, a fault beneath the western margin of the NMSZ.

Studies within the southern portion of the NMSZ reveal a minimum of three paleoliquefaction events in the 2000 years prior to 1811. These events are believed to have occurred 1500–2000 years ago, 1000–1200 years ago, and 600–800 years ago. At the northern end of the NMSZ, paleoearthquakes have been dated at approximately 1560 years ago, between 1100 and 1460 years ago, and between 980 and 1230 years ago.

The known faults of the study region with Quaternary offset (but without demonstrable Holocene offset) are all short (from 1 to 9 miles of mapped length) and are either located west of the current focus of seismic activity in the NMSZ or are small fault segments in the Fluorspar Area fault complex. Based on fault length, they may be capable of producing an earthquake of magnitude 6 or greater; however, based on lack of Holocene offset, they are thought to have a Pleistocene recurrence interval of more than 100,000 years. Faults in this group thus have a negligible contribution to the seismic hazard at PGDP.

3.2.2.2 Wabash Valley Seismic Zone

The WVSZ geographically includes portions of Kentucky, Indiana, and Illinois, and lies within the Illinois Basin. Although there is no current center of seismicity in the WVSZ, the area has a paleoseismic history of earthquakes of magnitude greater than 6. No Holocene fault displacement has been recognized in the WVSZ (Nelson et al. 1997).

Paleoliquefaction studies have been conducted along most of the major rivers of southern Indiana and Illinois. In these studies, riverbank sediments are studied for evidence of earthquake-induced sand dikes, sand sills, and sand blows. Based on paleoliquefaction studies of the WVSZ, investigators have recognized at least eight prehistoric earthquakes within the last 20,000 years that were strong enough to cause liquefaction (REI 1999). Of these eight earthquakes, six were probably greater than magnitude 6 and at least two are estimated to have exceeded magnitude 7. The largest prehistoric earthquake in the paleoliquefaction record occurred about 15 miles west of Vincennes, Indiana. This earthquake, is estimated to have had a magnitude greater than 7.5 and to have occurred 6100 ± 200 years ago.

3.3 REGIONAL GEOLOGY

The subsurface geologic units in the PGDP vicinity consist of approximately 350 ft of Cretaceous, Tertiary, and Quaternary sediments unconformably overlying Paleozoic bedrock. In the PGDP vicinity, the Cretaceous through late Tertiary sediments dip gently to the south-southwest toward the axis of the Mississippi Embayment and overlie northward-dipping Paleozoic bedrock. In stratigraphic order, Mississipian limestone bedrock is overlaid by a rubble zone, the Cretaceous McNairy Formation, the Paleocene Porters Creek Clay, undifferentiated Eocene sediments, and Pliocene and Pleistocene continental deposits (Fig. 3.6).

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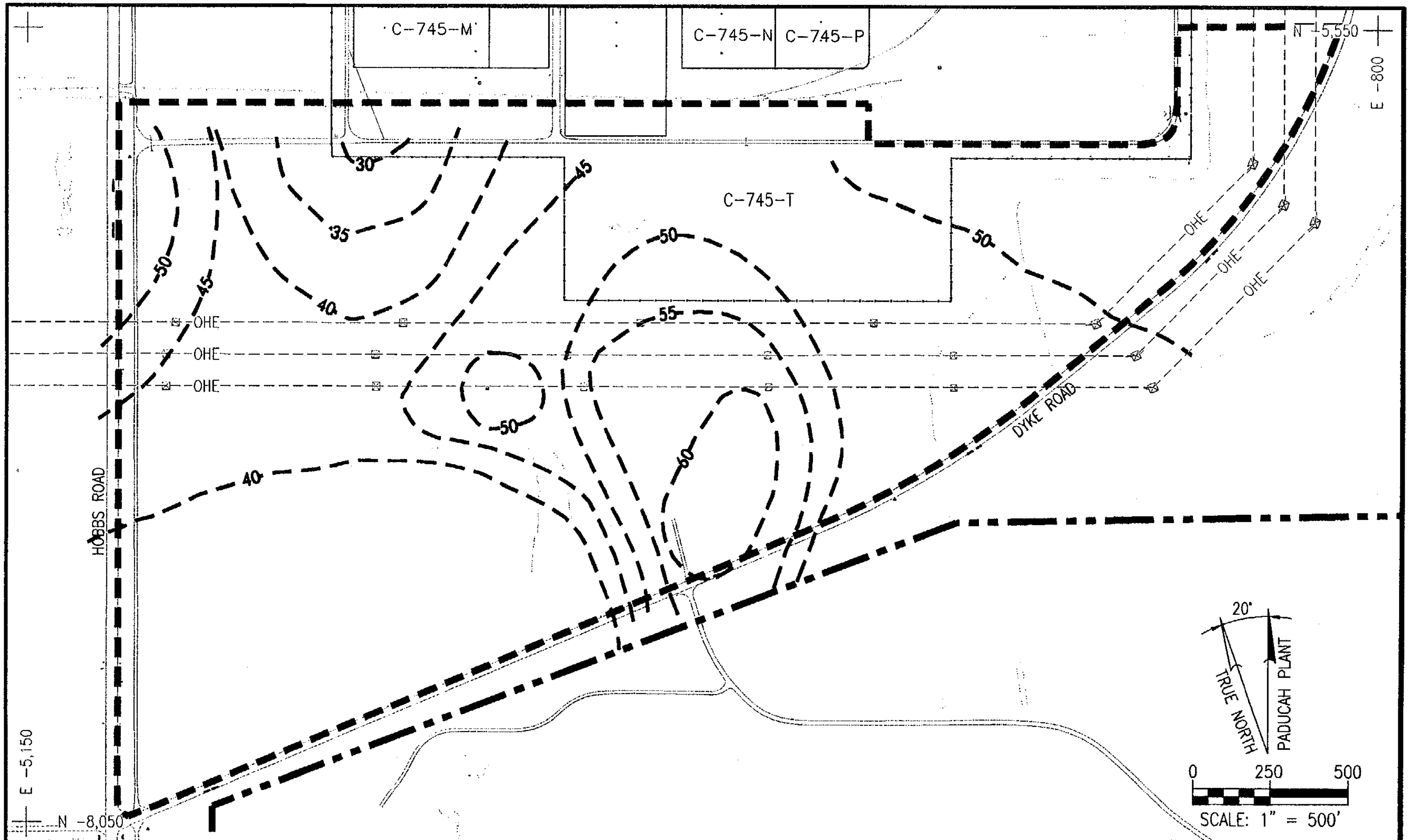

LEGEND:

- ASPHALT ROAD
- GRAVEL ROAD
- PGDP BOUNDARY
- SITE 3A BOUNDARY
- RAILROAD TRACKS
- FENCE LINE
- DEPTH TO TOP OF PORTERS CREEK CLAY (ft bgs)
- STREAM
- ELECTRICAL UTILITY LINES

SOURCE:

1.) BASEMAP FROM JACOBS ENGINEERING SWOU RI/FS, 1999.
2.) DEPTH TO TOP OF PORTER'S CREEK CLAY INFERRED FROM FIELD DATA.

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Fig. 3.6. Geologic columnar section of the PGDP vicinity.

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The erosion and subsequent fill of the ancestral Tennessee River valley during the Pleistocene is a primary factor controlling the shallow geologic units beneath PGDP. During the Pleistocene, the ancestral Tennessee River occupied a position close to the present-day course of the Ohio River. The southern edge of the former Tennessee River valley underlies the PGDP site. Figure 3.7 presents a schematic north-south cross-section of the geologic units extending from PGDP to the Ohio River.

At Site 3A, two deep boreholes were drilled during this Seismic Investigation. The first deep borehole (DB-01) was drilled to a total depth of 359 ft bgs using a Rotosonic drilling technique (Fig. 2.6). In addition, a natural gamma log was completed in DB-01. Figure 3.8 shows a comparison of the natural gamma and lithologic logs for borehole DB-01. Stratigraphic interfaces between soil layers were observed in the continuous Rotosonic core. The second deep borehole (DB-02) was drilled to a total depth of 400.3 ft bgs using mud rotary techniques (Fig. 2.6). No sampling was done in borehole DB-02 within the McNairy Formation; however, seismic velocities were measured in DB-02 that were used to interpret results of the p-wave survey. These results are described in the following paragraphs.

3.3.1 Bedrock

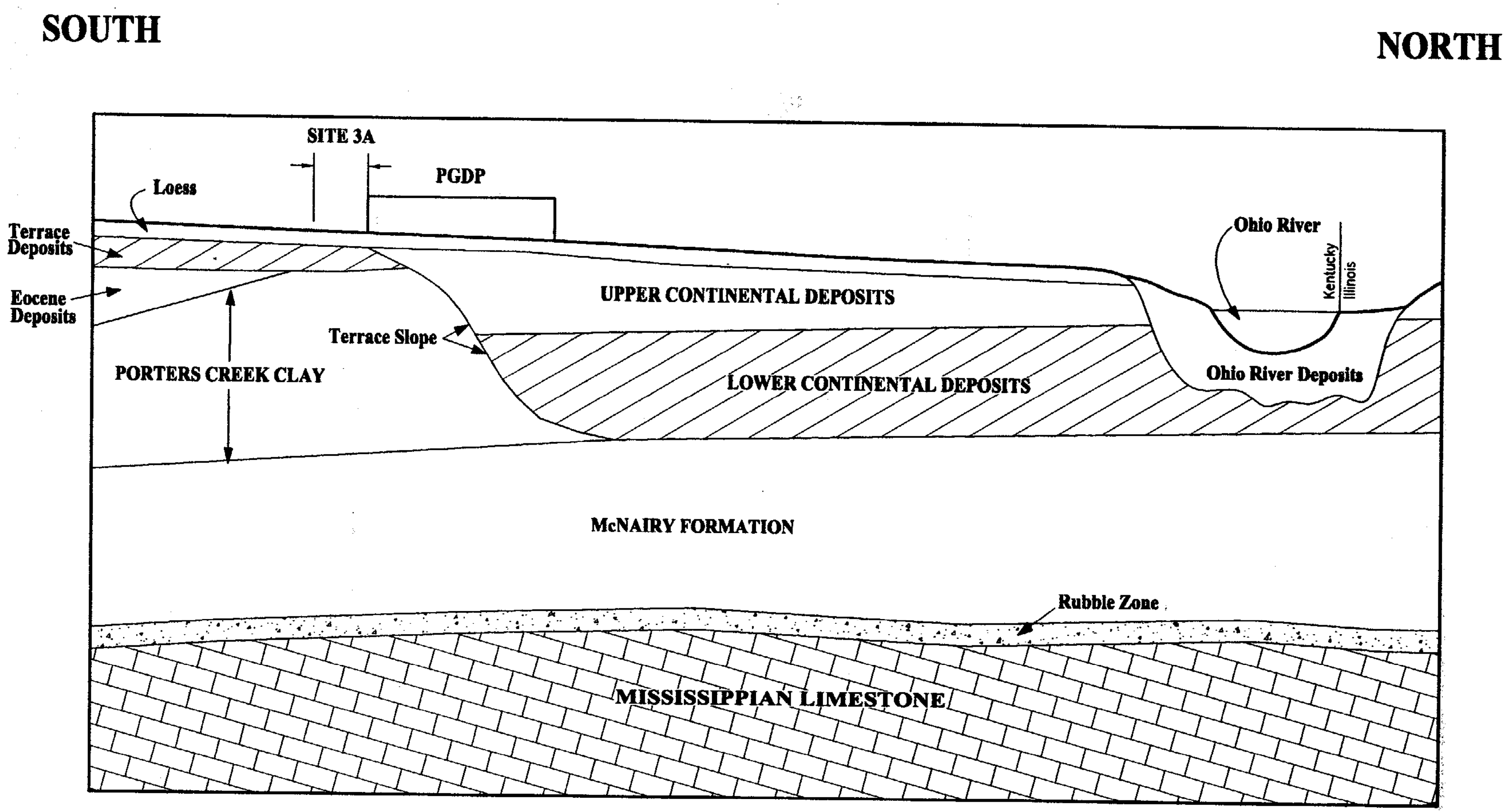
Limestone, which is believed to be Mississippian-age Warsaw Limestone, subcrops beneath PGDP. Deep borings at PGDP have typically encountered limestone bedrock at depths of approximately 335 to 350 ft bgs. A rubble zone, which consists of a 5- to 20-ft-thick layer of subangular chert and silicified limestone fragments, immediately overlies bedrock at PGDP.

At Site 3A, bedrock was encountered at a depth of 400 ft bgs in borehole DB-02, which is deeper than typically encountered at PGDP. The presence of a rubble zone on the bedrock surface could not be determined at Site 3A because drilling was stopped upon encountering hard rock material. Results of the p-wave survey (Appendix B) provide an indirect measure of depth to bedrock across the site. The top of the bedrock interface has been inferred from the interpreted instantaneous phase sections presented for each seismic reflection survey line in Appendix B. The p-wave survey results represent depth as two-way time intervals in msec. By calibrating these profiles for the actual measured bedrock surface at DB-02, an interpreted depth to bedrock has been inferred (Fig. 3.9). Calibration was performed using an average seismic velocity of 4600 feet per second (fps) for the sediments overlying the limestone bedrock. This average seismic velocity is relatively consistent with results of the seismic velocity log measured for DB-02 (Appendix E).

Based on these results, the depth to bedrock at Site 3A ranges from approximately 325 ft bgs in the northwest corner of the site to as much as 425 ft bgs in the central and southeastern portion of the site. By superimposing the land surface topography over the depth to bedrock interface data, an approximation of the bedrock surface elevation can be developed, as shown in Fig. 3.10. The bedrock surface ranges in elevation from a high of approximately 50 ft above msl in the northwest corner of the site to a low of 40 ft below msl in the central and southeastern portion of the site.

3.3.2 McNairy and Clayton Formations

Unconsolidated deposits of the Upper Cretaceous McNairy Formation overlie the rubble zone. This formation is composed of interbedded and interlensing sand, silt, and clay deposits of marine origin. The sands are typically well-sorted, fine-grained, micaceous, and commonly glauconitic (i.e., common rock-forming minerals). Near PGDP, the McNairy Formation can be subdivided into three lithologic members: (1) a 100- to 120-ft-thick sand-dominant lower member; (2) a 55- to 60-ft-thick middle member composed predominantly of silty and clayey fine sand; and (3) a 60- to 70-ft-thick upper member consisting of interbedded sands, silts, clays, and occasional gravels. Deposits of the Clayton Formation overlie the



Not to scale

Fig. 3.7. Schematic geologic cross section of the PGDP area.

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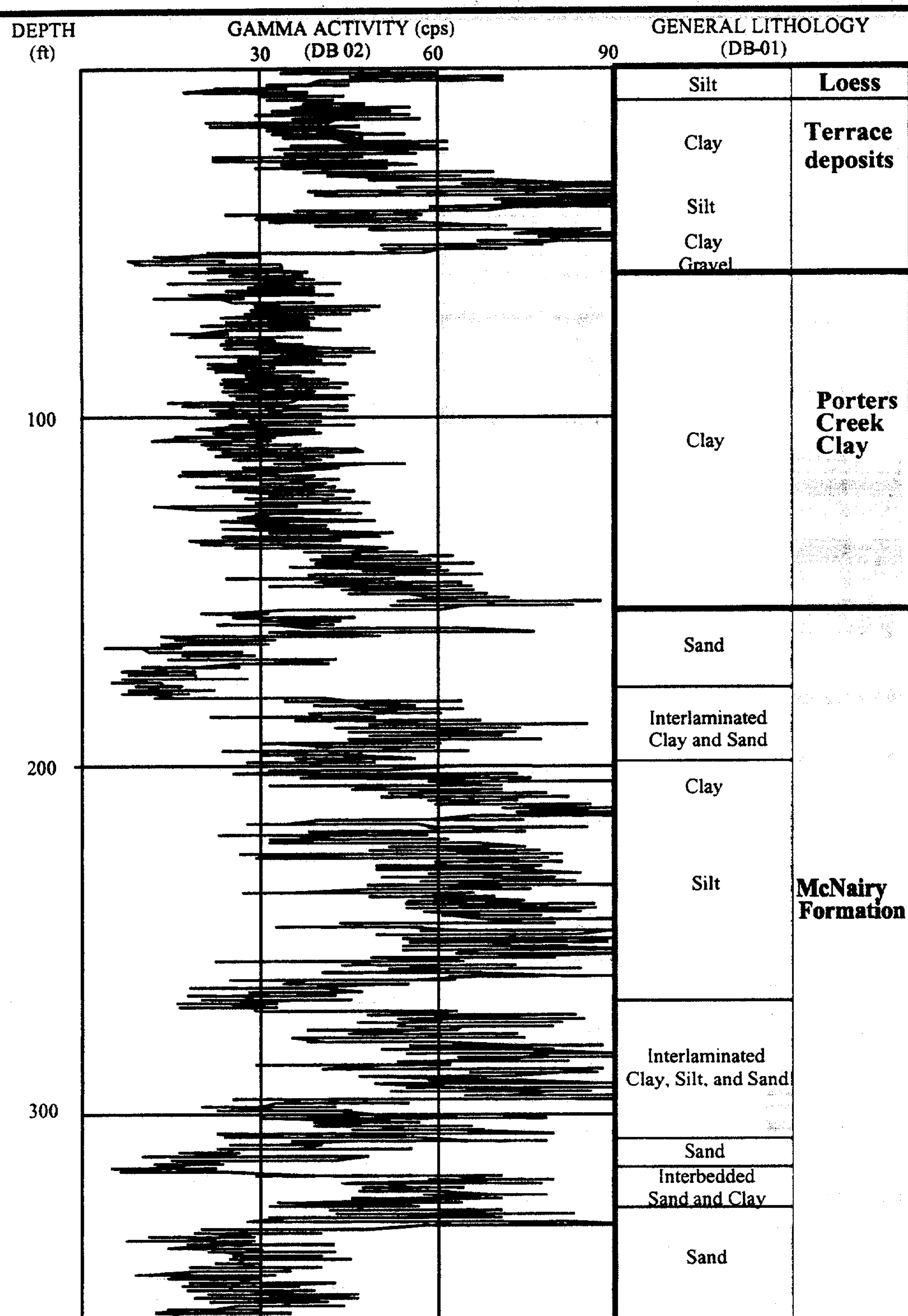

Not to scale

Fig. 3.8. Borehole DB-01 natural gamma and lithology logs.

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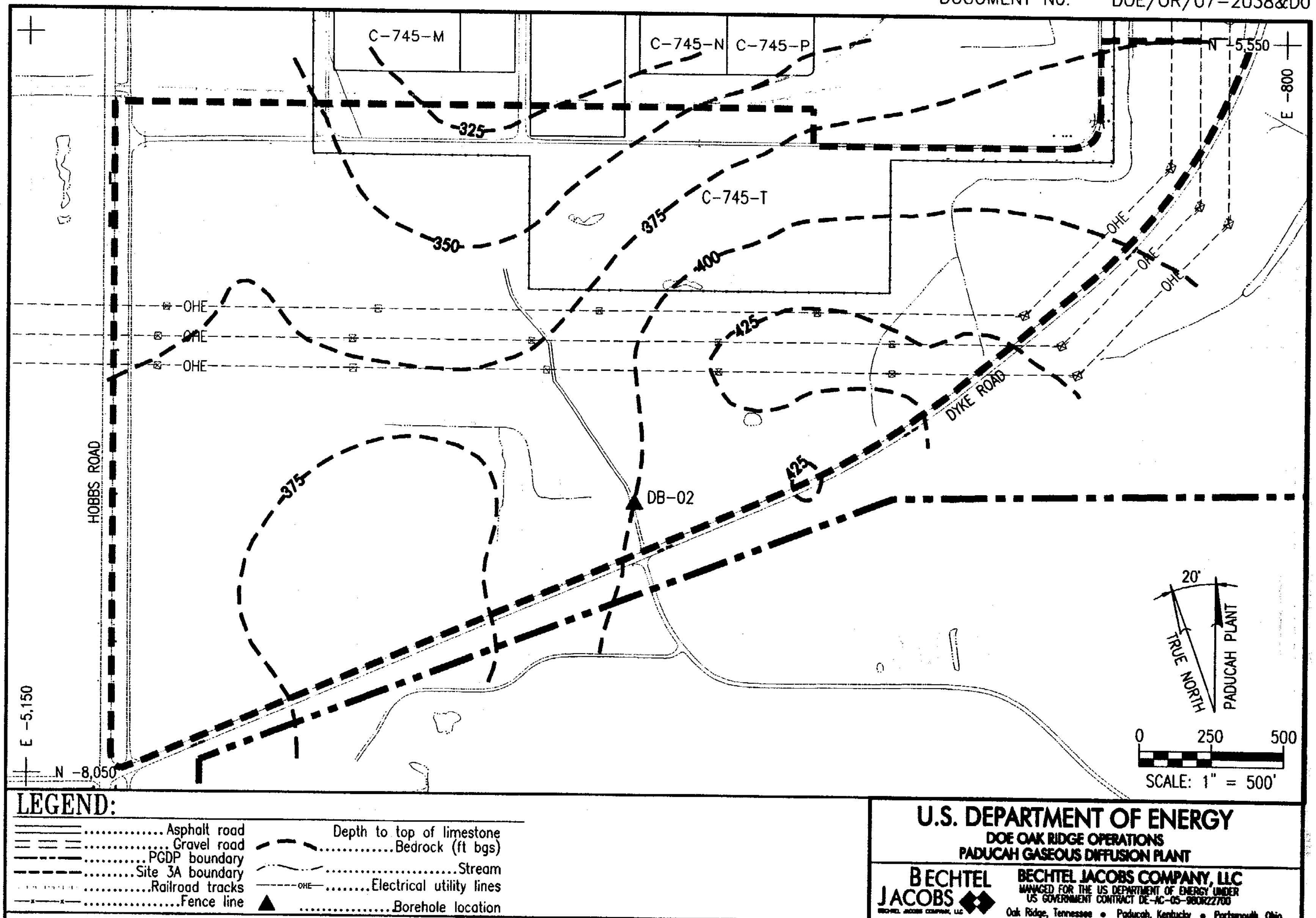

Fig. 3.9. Depth to top of limestone bedrock (ft bgs).

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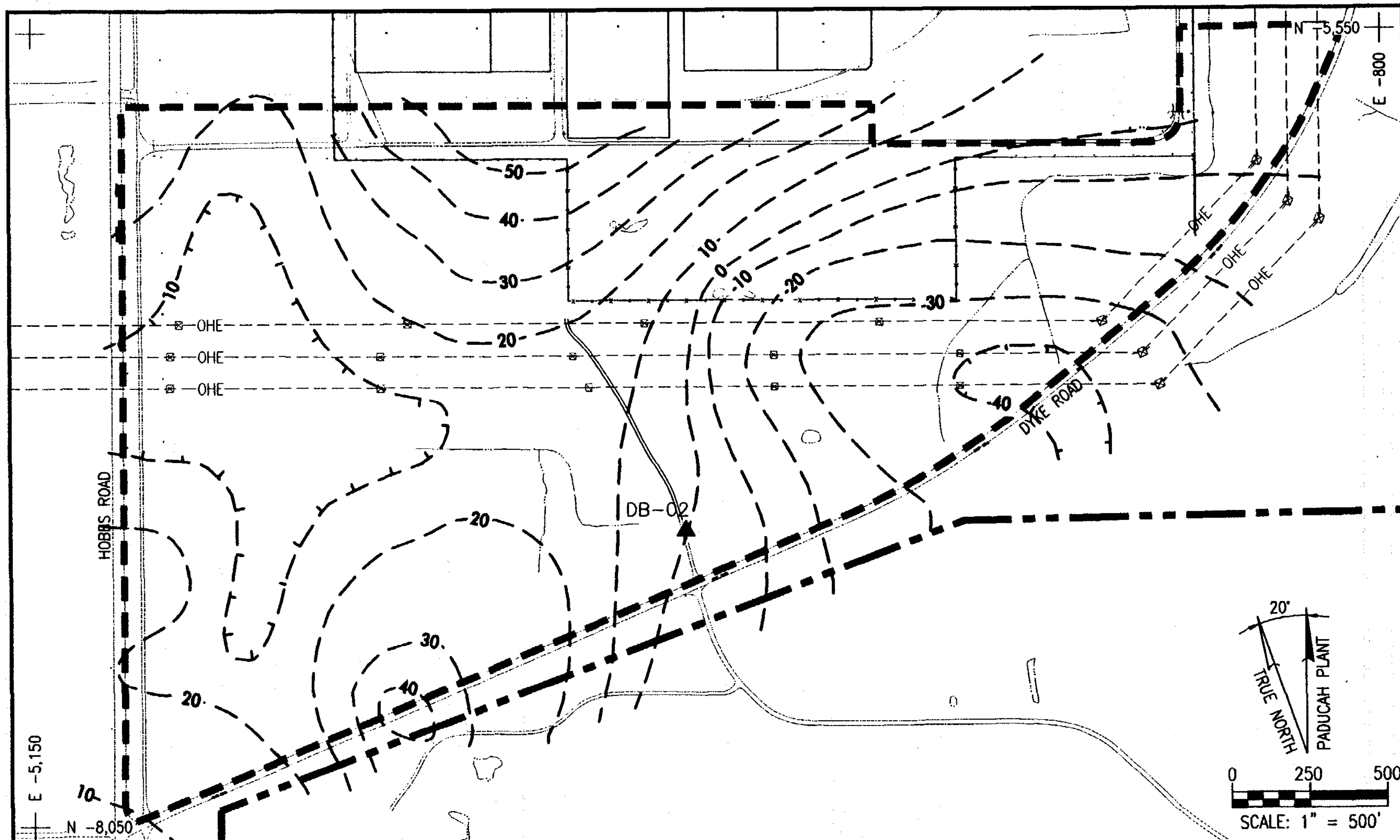

LEGEND:

Asphalt road		Elevation of top of limestone Bedrock (ft msl)	
Gravel road		Stream	
PGDP boundary		Electrical utility lines	
Site 3A boundary		Borehole location	
Railroad tracks			
Fence line			

Fig. 3.10. Elevation of top of limestone bedrock (ft msl).

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McNairy Formation. Because of difficulties in distinguishing between the Clayton and McNairy Formations at PGDP, these lithologies have been grouped together and termed the McNairy Formation. Total thickness of the McNairy Formation is typically 220 to 250 ft.

At Site 3A, the upper sand facies of the McNairy Formation was encountered in Rotosonic borehole DB-01 at a depth of 155 ft bgs. A predominantly silt and clay facies was encountered below a depth of approximately 180 ft bgs, and a lower sand facies was encountered below 255 ft bgs. The total thickness of the McNairy Formation at DB-01 was 245 ft, which is somewhat greater than that typically encountered at PGDP due to the lower bedrock interface at Site 3A.

The McNairy Formation interface was interpreted also in the p-wave survey data (Appendix B). However, based on measured seismic velocities in DB-02, the reflection interface presented on the interpreted instantaneous phase sections in Appendix B likely corresponds to the top of the lower sand facies, not the upper sand facies. The top of the upper sand facies is not apparent in the seismic reflection survey data.

3.3.3 Porters Creek Clay

The Paleocene Porters Creek Clay occurs in southern portions of the DOE property as a massive, glauconitic clay with lesser interbeds of sand overlying the McNairy Formation. A terrace slope of the ancestral Tennessee River completely cuts through the thickness of the Porters Creek Clay under the south end of PGDP (Fig. 1.1). The Porters Creek Clay is approximately 100 ft thick immediately southwest of PGDP, but is absent or present only as thin isolated remnants, to the north of the terrace slope.

Outcrops of the Porters Creek Clay on DOE property are limited to a few isolated locations in the bed of Bayou Creek and its tributaries. However, borehole data are sufficient to show that the top of the Porters Creek Clay south of PGDP has significant topographic relief. Immediately south and west of PGDP, the high elevation of the top of the Porters Creek Clay limits the development of a shallow groundwater system in that area. A greater depth to the top of the Porters Creek Clay to the east of PGDP permitted deposition of a relatively permeable Pliocene gravel near the surface.

At Site 3A, the Porters Creek Clay was encountered in several SPT borings, SCPT soundings, and DPT boreholes, although in some cases the SCPT or DPT probes encountered refusal in overlying sediments before reaching the Porters Creek Clay. The Porters Creek Clay was well defined in the s-wave survey along survey Line 3S, but was less clearly defined along survey Line 2S. As a result, information on the depth to the top of the Porters Creek Clay is sketchy in some areas, particularly in the eastern portion of the site (Fig. 3.11). Where evident, the Porters Creek Clay was generally found at depths of 30 to 60 ft bgs. At the deeper Rotosonic borehole (DB-01), which penetrated beneath the Porters Creek Clay into the McNairy Formation, the Porters Creek Clay was found to be 95 ft thick. Similarly, at borehole Z-1 (installed in a previous investigation near the northwest corner of Site 3A), the Porters Creek Clay was encountered between depths of 35 and 125 ft bgs, for a thickness of approximately 90 ft.

Figure 3.12 presents an approximation of the surface elevation of the top of the Porters Creek Clay, developed by superimposing the land surface topography with the depth to the top of the Porters Creek Clay interface data. The surface of the Porters Creek Clay at Site 3A ranges in elevation from a high of approximately 365 ft msl in the southern corner of the site to a low of 330 ft msl in the northwest corner of the site. A lower area is also present in the central and southeastern portion of the site, corresponding to a lower area present in the top of the limestone bedrock.

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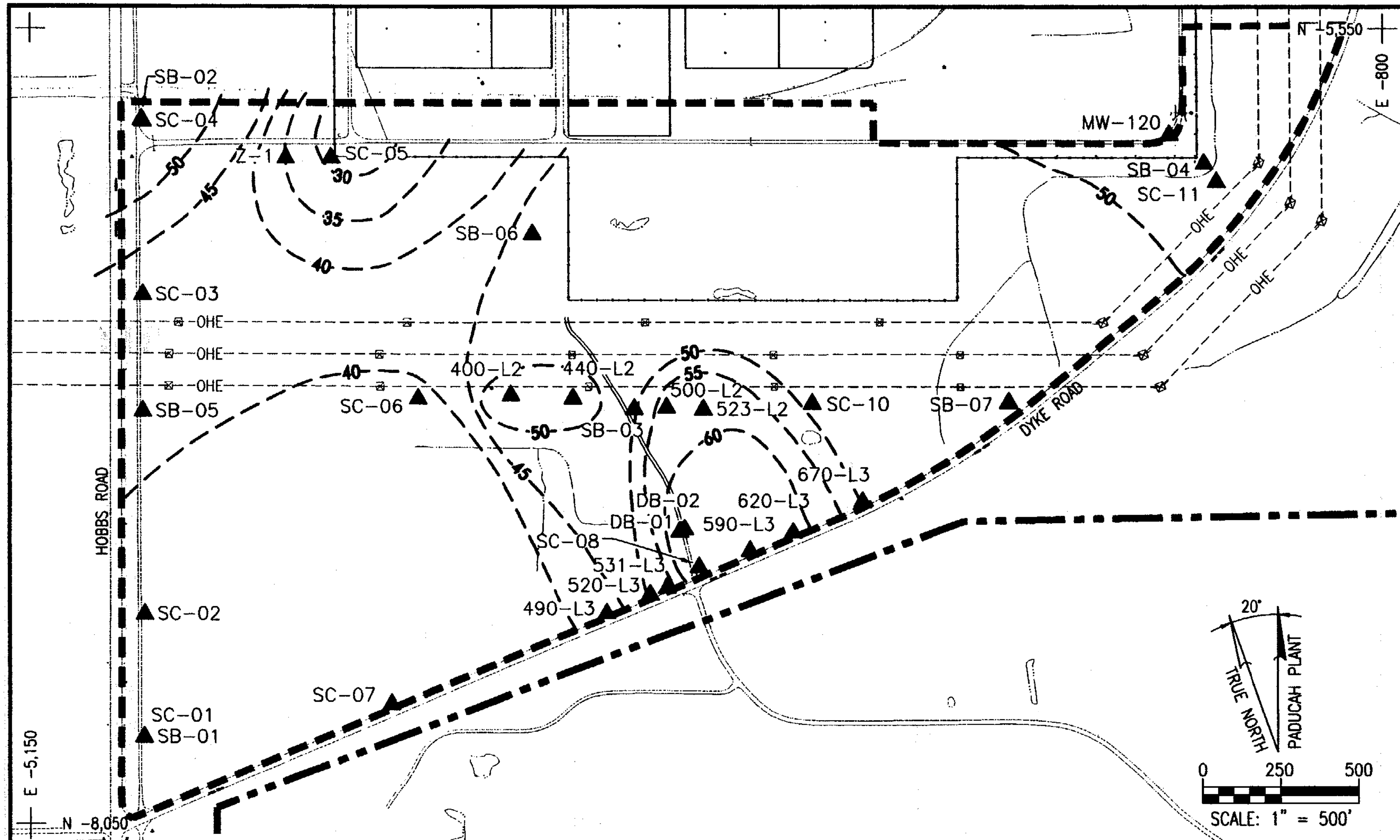

LEGEND:

══════	Asphalt road	⁻ ⁻ ⁻	Depth to top of Porters Creek Clay (ft bgs)
═ ═ ═	Gravel road	~~~	Stream
— — —	PGDP boundary	---OHE---	Electrical utility lines
■ ■ ■	Site 3A boundary	▲	Borehole location
· · · ·	Railroad tracks		
—x—x—	Fence line		

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Fig. 3.11. Depth to top of Porters Creek Clay (ft bgs).

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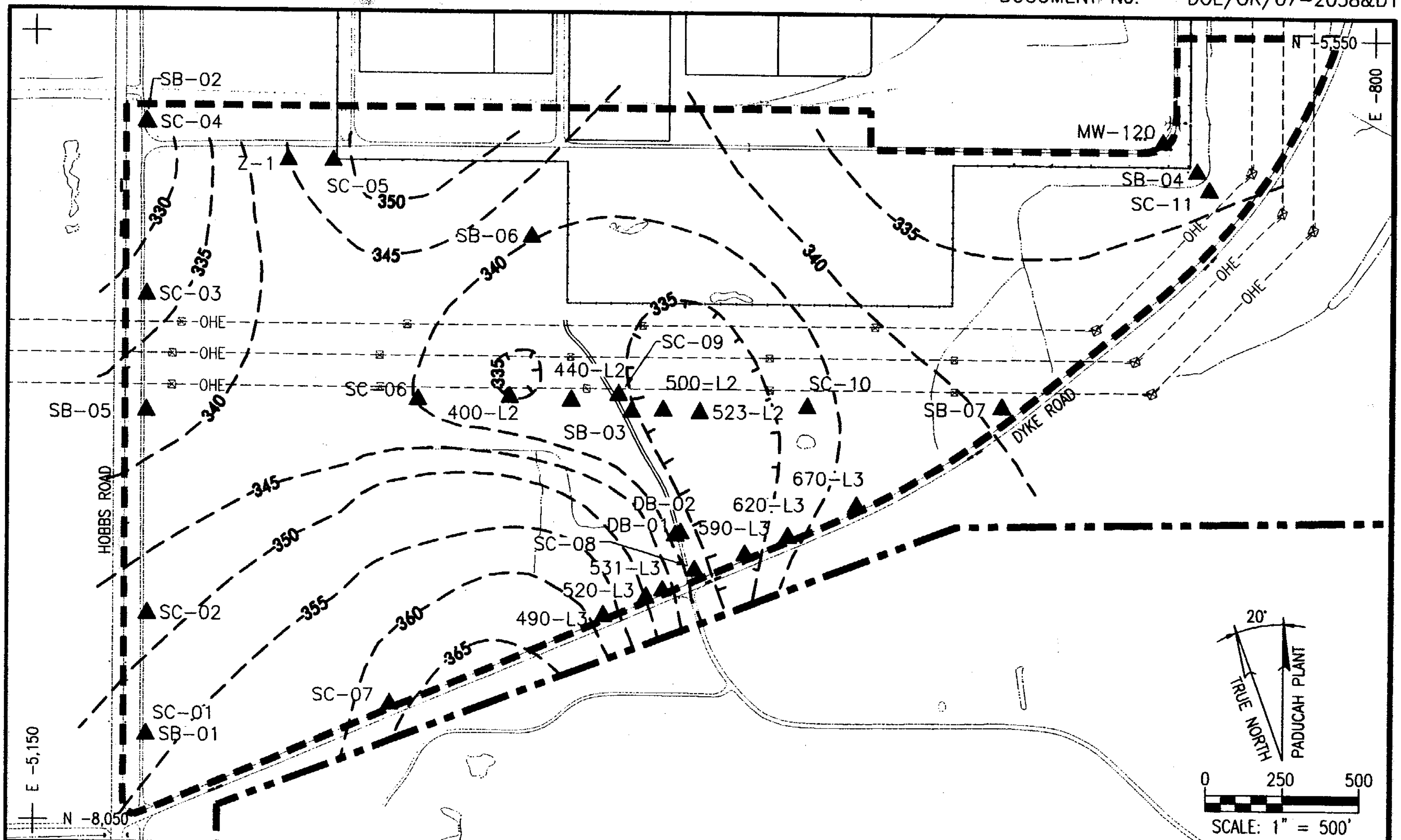

LEGEND:

Asphalt road	Elevation of top of Porters Creek Clay (ft msl)
Gravel road	Stream
PGDP boundary	Electrical utility lines
Site 3A boundary	Borehole locations
Railroad tracks	
Fence line	

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Fig. 3.12. Elevation of top of Porters Creek Clay (ft msl).

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3.3.4 Eocene Deposits

Eocene sands, silts, and clays overlie the Porters Creek Clay south of PGDP. Researchers have not attributed these sediments to a specific formation. The thickness of the Eocene sediments approaches zero near the terrace slope and increases southward to more than 100 ft. Eocene deposits do not underlie PGDP north of Site 3A. It is unlikely that Eocene sediments are present at Site 3A, because the gravels encountered at Site 3A are extensions of Pliocene gravels or are secondary deposits derived from erosion of the Pliocene gravels.

3.3.5 Continental Deposits

Pliocene and Pleistocene deposits unconformably overlie the Cretaceous through Eocene strata in the vicinity of PGDP. The Pliocene deposits consist of lobes of poorly sorted, silty sand and gravel that occur south of PGDP. These sediments represent an alluvial fan deposit that covered all of western Kentucky and parts of Tennessee and Illinois during the Pliocene Epoch. Beginning under the south end of PGDP and extending north beyond the Ohio River, a thick sequence of Pleistocene continental deposits fills the buried valley of the ancestral Tennessee River. This sediment package consists of a basal sand and gravel member, the Lower Continental Deposits, and an overlying finer-textured lithofacies, the Upper Continental Deposits. Where fully developed, the Upper Continental Deposits include a bottom sand unit overlaid by a thick silt and clay interval containing at least two horizons of sand and gravel.

Lower Continental Deposits. Pleistocene sand and gravel units, collectively averaging 30 ft thick, underlie most of PGDP and the northern portion of the DOE property. Depth to the top of this lower member is approximately 60 ft. The matrix is characteristically medium to coarse sand and chert gravel of variable sorting. This unit is not present at Site 3A.

Upper Continental Deposits. The upper (Pleistocene) sediments include a wide variety of textures within three depositional series. A basal sand unit is generally present, and has a fining upward texture, becoming siltier toward the top of the basal unit. A middle unit occurs generally as a silty clay or clayey silt, and has a thickness that varies widely from less than 10 to 40 ft. Sand and gravel deposits define an upper unit. Texture and sorting are widely variable among the sand and gravel deposits. Other than the broad lens character of some sand and gravel units, the Upper Continental Deposits do not contain recognizable bedding features. Gradational textural changes are common. Silt and clay facies typically are mottled and contain frequent vertical traces filled with lighter colored silt or clay. At PGDP, the Upper Continental Deposits have become synonymous with the Upper Continental Recharge System (UCRS), which is a zone of predominantly vertical groundwater flow that is confined to the shallow deposits of the ancestral Tennessee River valley beneath the plant.

Terrace Deposits. At Site 3A, continental deposits directly overlie the Porters Creek Clay on top of the Porters Creek Clay terrace. These Terrace Deposits are near-synchronous with the Upper Continental Deposits and are similar to, and may be locally continuous with, the Upper Continental Deposits. The term "Terrace Deposits" is used to differentiate the groundwater flow systems; groundwater flow in the Terrace Deposits is predominantly lateral, confined below by the Porters Creek Clay. The Terrace Deposits consist of clay, silt, sand, and gravel deposits similar to those of the Upper Continental Deposits. Gravel deposits, particularly at depth, may be Pliocene-age or derived from Pliocene-age gravels present south of the site. These gravel units typically occur below a depth of 15 to 20 ft at Site 3A. The cumulative thickness of the Terrace Deposits varies from 20 to as much as 50 ft.

3.3.6 Surficial Deposits/Soils

Silt of the Pleistocene Peorian Loess and an older unit tentatively identified as the Roxanna Loess covers sediments both north and south of the buried terrace slope (DOE 1997). The loess deposit is virtually indistinguishable from a silt facies of the Upper Continental Deposits. Loess typically is 10 to 15 ft thick beneath most of PGDP; however, construction activities have excavated the loess or replaced the loess with fill material in many areas. At Site 3A, the loess is approximately 15 to 20 ft thick.

Radiocarbon age dating of the surficial deposits at Site 3A was completed as part of this Seismic Investigation. Results are discussed in Sect. 6.2.3. The radiocarbon dates confirm that the loess is generally late Pleistocene in age with ^{14}C dates ranging from 13,540 to 15,620 years before present, where "present" is defined as 1950 A.D. (BP). In some areas, there are younger alluvial deposits at shallow depths that fill former erosional features incised in the loess. The uppermost 3 to 4 ft may represent recent alluvium of Holocene-age deposition or a composite age of organic matter due to bioturbation (e.g., animal burrows or tree roots). These shallow Holocene-age alluvial deposits yielded ^{14}C dates ranging from 3770 to 4190 years BP.

Soils of the area are predominantly silt loams that are poorly drained, acidic, and have little organic content. Six soil types are associated with PGDP as mapped by the Natural Resources Conservation Service, formerly the Soil Conservation Service (USDA 1976). The dominant soil types, the Calloway and Henry silt loams, consist of nearly level, somewhat poorly drained to poorly drained soils that formed in deposits of loess and alluvium. These soils tend to have low organic content, low buffering capacity, and acidic pH ranging from 4.5 to 5.5. The Calloway and Henry series have a fragipan horizon, a compact and brittle silty clay loam layer that extends from 26 inches bgs to a depth of 50 inches or more. The fragipan reduces the vertical movement of water and causes a seasonally perched water table in some areas. At Site 3A, the Henry silt loam is the predominant soil type, although Calloway silt loam and Grenada silt loam are present along the southeastern portion of the site near Dyke Road (USDA 1976).

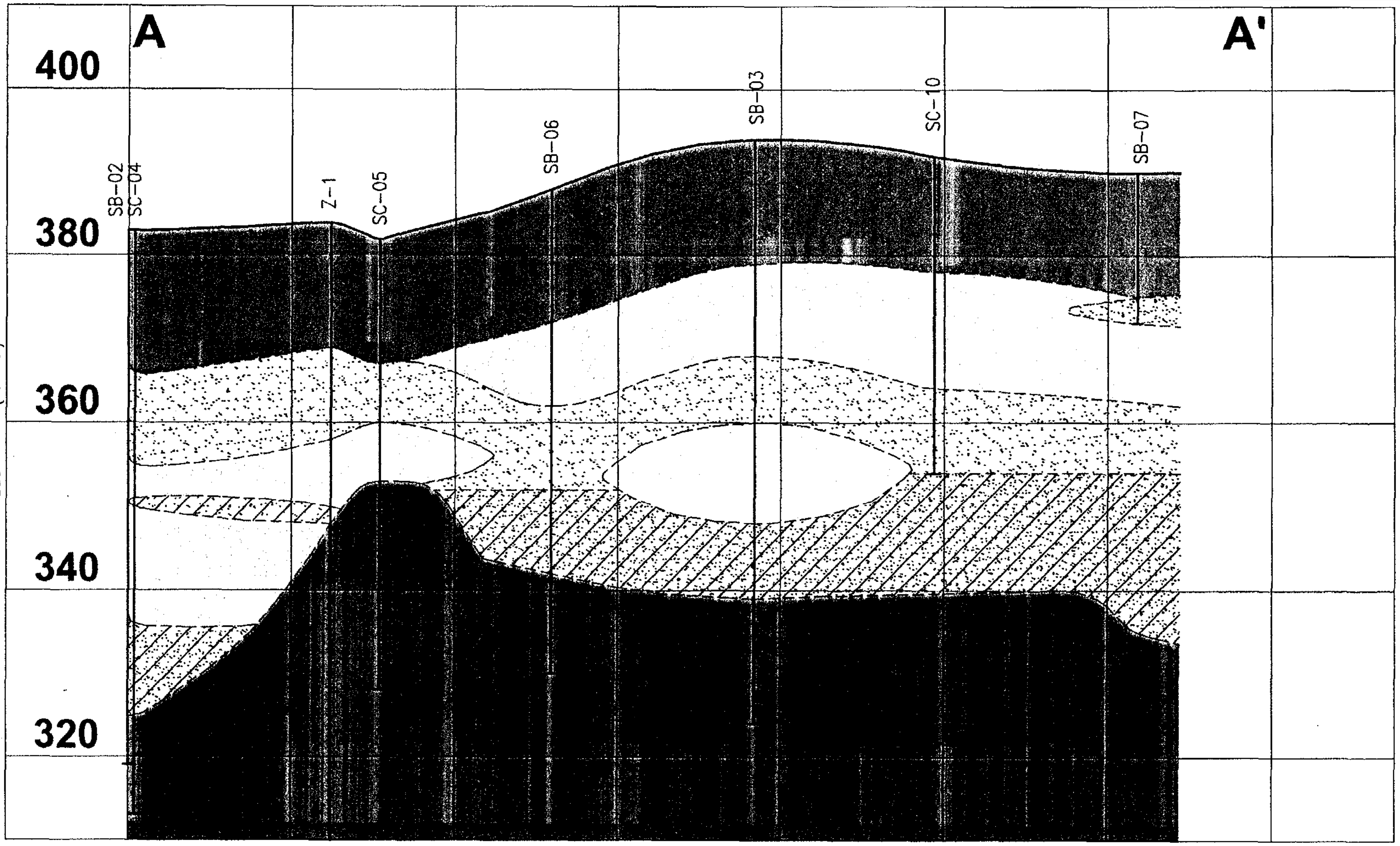
3.4 SUMMARY OF SITE 3A STRATIGRAPHY

Stratigraphy at Site 3A has been defined through evaluation of the results of the seismic reflection surveys, SCPT soundings, DPT boreholes, and mud rotary boreholes. Geologic cross-sections through the site are presented in Figs. 3.13 through 3.16.

Limestone bedrock was encountered in the deep mud rotary borehole DB-02 at a depth of 400 ft bgs. The bedrock surface varies between 325 to as much as 425 ft bgs, based on interpretations of the p-wave survey data. The elevation of the bedrock surface ranges between 50 ft above msl and 40 ft below msl.

The McNairy Formation was encountered overlying bedrock. The formation was encountered in the deep Rotosonic and mud rotary boreholes, but was not encountered in any of the shallow boreholes. The upper sand facies of the McNairy Formation was encountered in Rotosonic borehole DB-01 at a depth of 155 ft bgs. This upper facies is a fine to very fine grained, poorly graded sand, light greenish gray to grayish brown. The sand is predominantly quartz, but contains abundant glauconite. A predominantly silt and clay facies was encountered below a depth of approximately 180 ft bgs. This middle facies consists of a firm, plastic dark gray to black clay overlying a firm gray micaceous silt of low to medium plasticity. A lower sand facies was encountered below 255 ft bgs. This lower facies is a fine to very fine, poorly graded sand, light gray, with interbedded and laminated silt and clay lenses. The total thickness of the McNairy Formation at DB-01 was 245 ft; the thickness of the unit is expected to be relatively constant across the site based on the p-wave survey results.

Six soil zones have been delineated at Site 3A for purposes of interpreting site-specific stratigraphy and geotechnical soil properties. These six soil zones are defined in the following paragraphs.



LEGEND:

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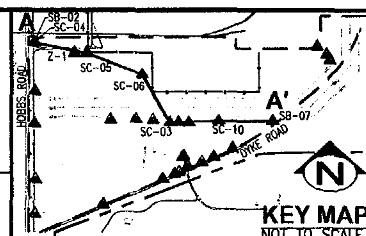
Soil zone 2	
Soil zone 3	
Soil zone 4	HORIZ. SCALE: 1"=500' (0 250 500)
Soil zone 5	
Soil zone 6	

Fig. 3.13. Cross section A-A'.



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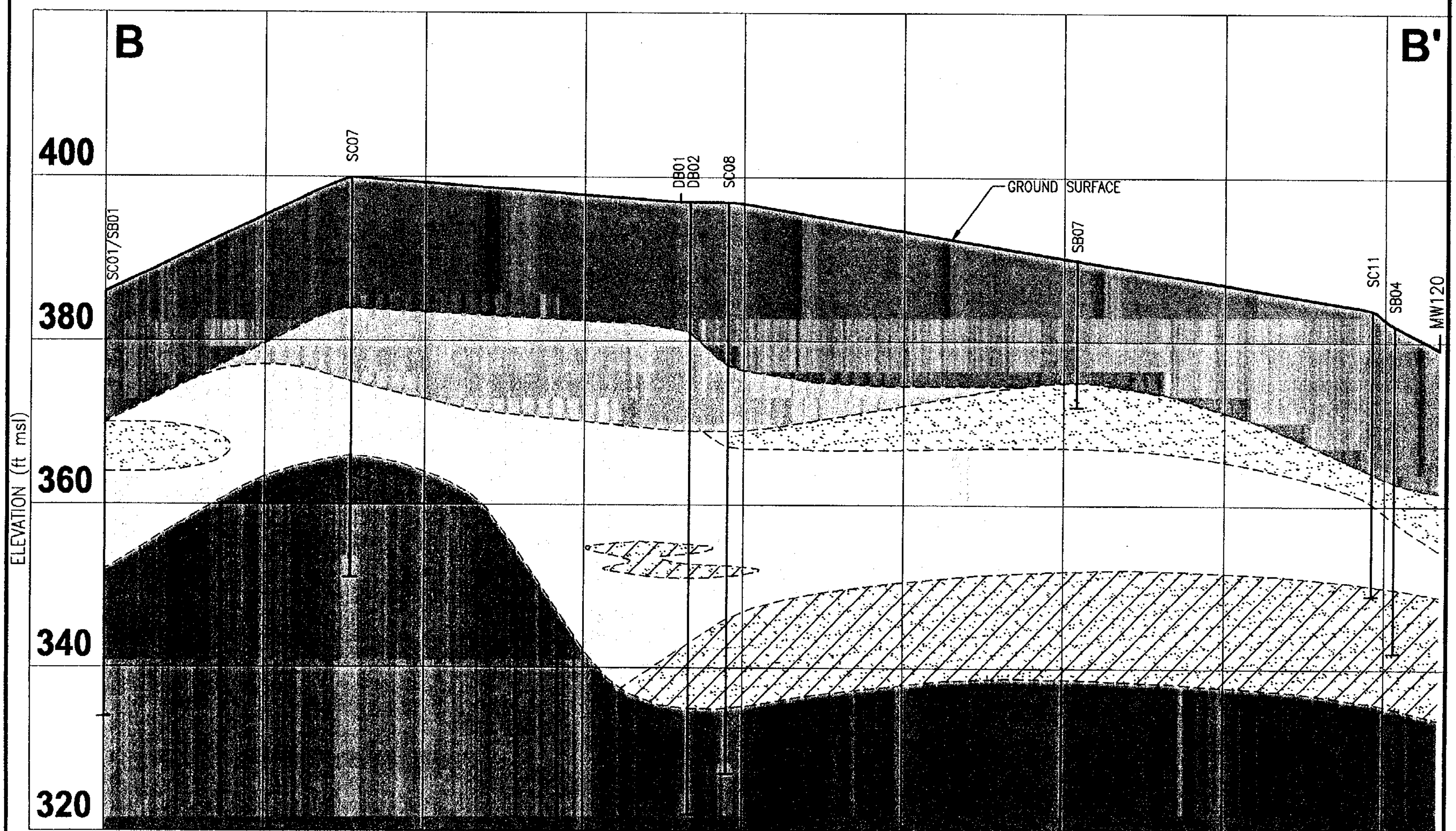

LEGEND:

- Soil zone 1
- Soil zone 2
- Soil zone 3
- Soil zone 4
- Soil zone 5
- Soil zone 6

0 5 10 20
VERT. SCALE: 1"=20'

0 250 500
HORIZ. SCALE: 1"=500'

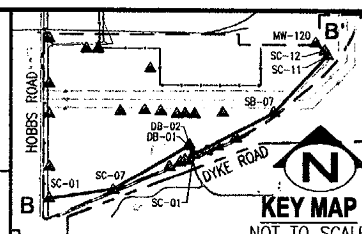

Fig. 3.14. Cross Section B-B'.

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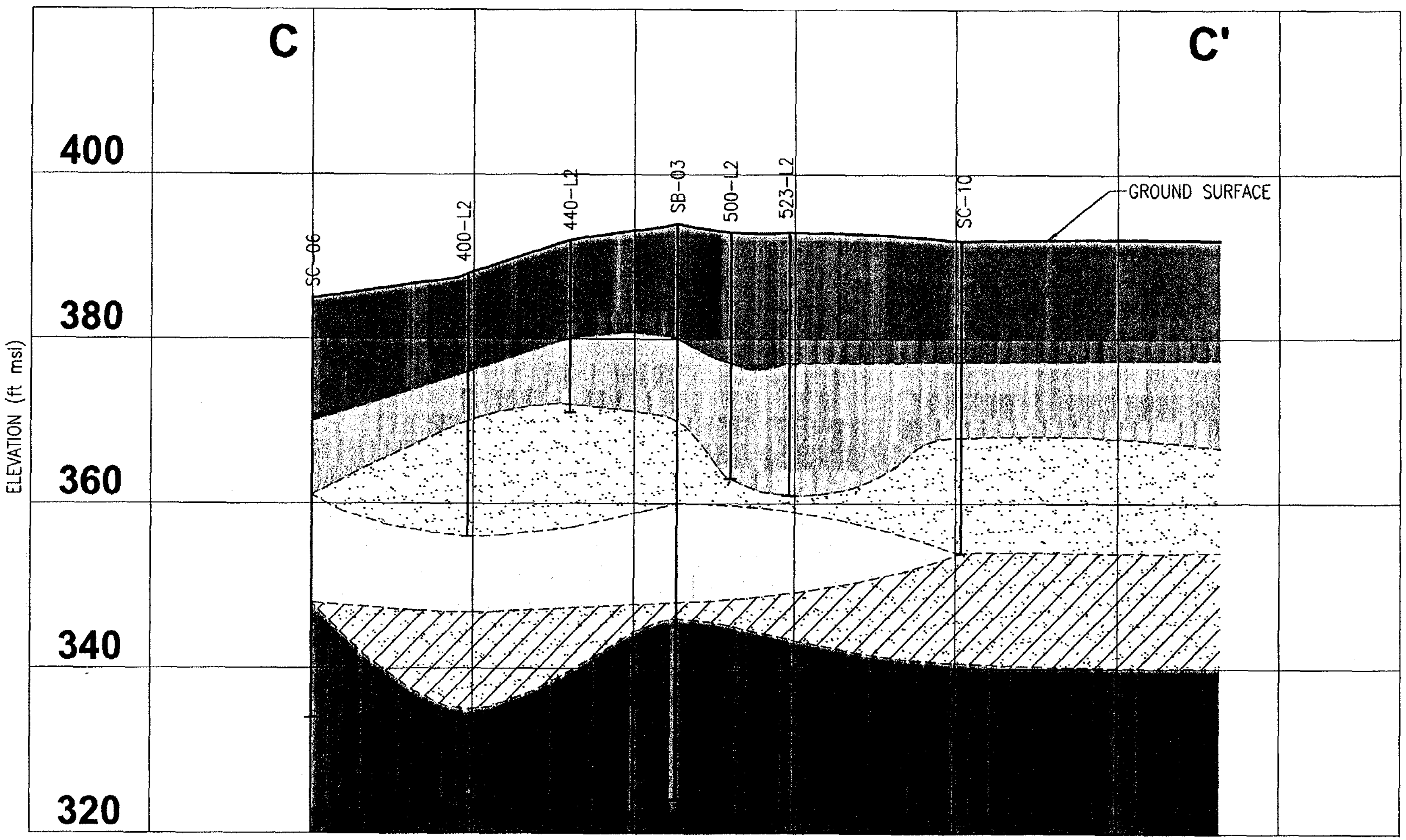


LEGEND:

- ....Soil zone 1
- ....Soil zone 2
- ....Soil zone 3
- ....Soil zone 4
- ....Soil zone 5
- ....Soil zone 6

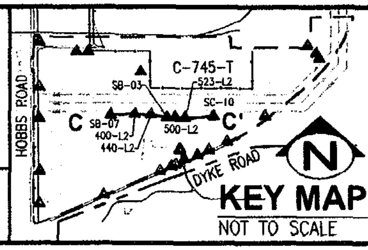
0 5 10 20

VERT SCALE: 1"=20'

0 100 200 300

HORIZ. SCALE: 1"=300'

Fig. 3.15. Cross section C-C'.



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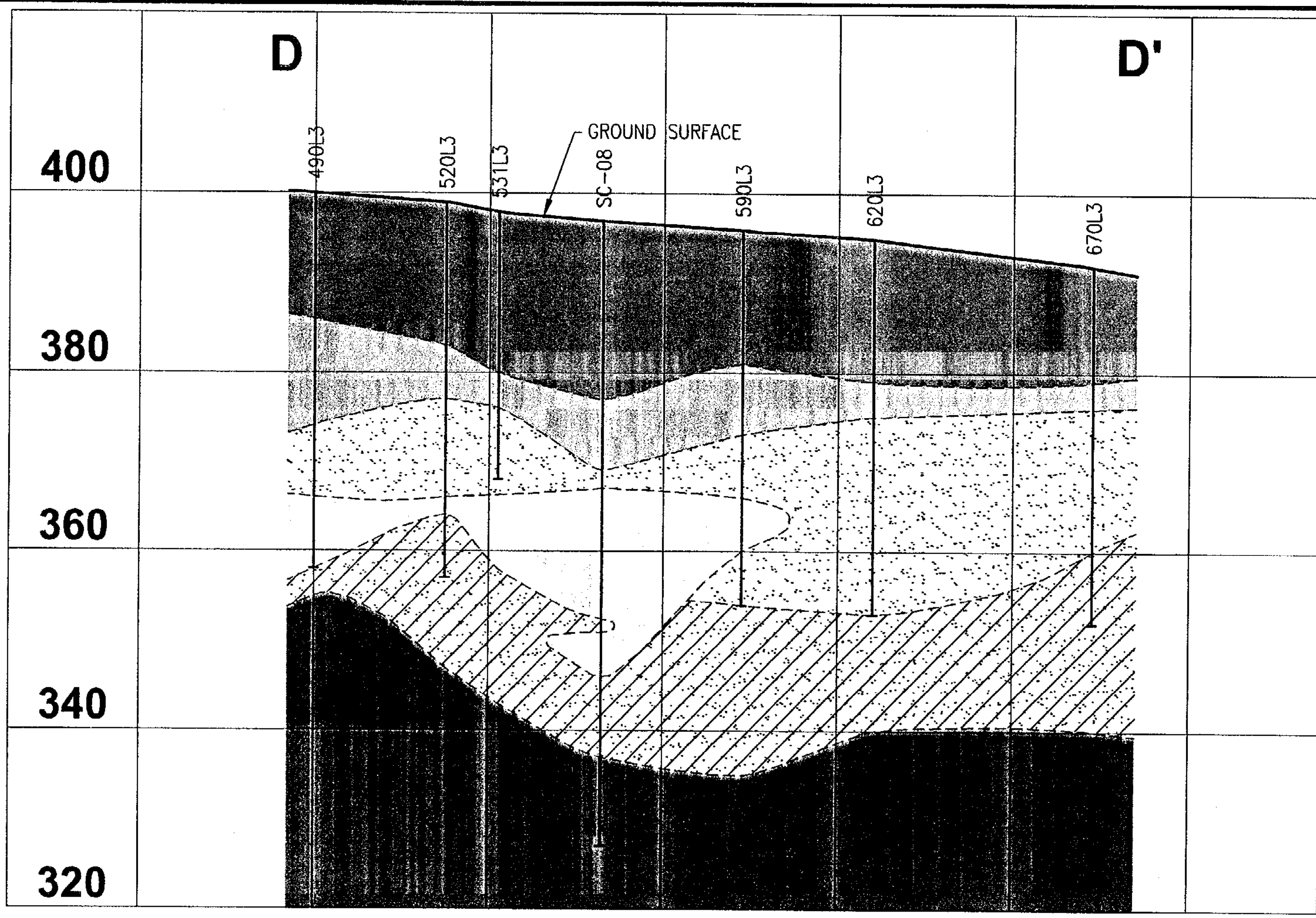


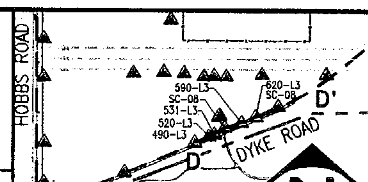
LEGEND:

- ....Soil zone 1
- ....Soil zone 2
- ....Soil zone 3
- ....Soil zone 4
- ....Soil zone 5
- ....Soil zone 6

0 5 10 20
VERT SCALE: 1"=20'

0 100 200
HORIZ. SCALE: 1"=200'

Fig. 3.16. Cross section D-D'.



KEY MAP
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Soil zone 6 is the Porters Creek Clay Formation. The Porters Creek Clay was encountered in several SPT borings, SCPT soundings, and DPT boreholes, and was partially defined in the s-wave survey. Where evident, the top of the Porters Creek Clay was generally found at depths of 30 to 60 ft bgs. At borehole DB-01, which penetrated beneath the Porters Creek Clay, the unit was found to be 95 ft thick and at borehole Z-1 (installed in a previous investigation near the northwest corner of Site 3A), the unit was found to be 90 ft thick. The surface of the Porters Creek Clay at Site 3A ranges in elevation from 330 to 365 ft msl. The Porters Creek Clay is described in boring logs as a firm to hard, very dark gray to black laminated silt and clay of low plasticity. Subsequent results of index property testing (Sect. 3.7.2.1) have shown that the Porters Creek Clay is actually a silty sand of high plasticity. The unit is designated as soil zone 6 in the cross-sections in Figs. 3.13 through 3.16.

Soil zones 2 through 5 represent the Terrace Deposits. The Terrace Deposits are typically present below a depth of 15 to 20 ft bgs. The thickness of the deposits varies from 20 to as much as 50 ft. The deposits vary considerably across the site in texture and stiffness and are present in beds that are not laterally continuous across the site. Considerable variation and gradual transition between the soil zones is common. The following generally describe the soil zones designated for the Terrace Deposits at Site 3A.

Soil zone 5 consists of a lower sand unit generally present below a depth of 32 ft bgs and directly overlying the Porters Creek Clay. The unit consists generally of a fine to medium grained, poorly graded sand, light brown, grading downward to a well graded sand and gravel, yellowish brown.

Soil zone 4 consists of interbedded silts and clays with occasional poorly graded sand lenses. The deposits typically overly zone 5 deposits, but directly overly the Porters Creek Clay at SB-05. The soils are generally firm, medium plasticity, light gray to brownish yellow.

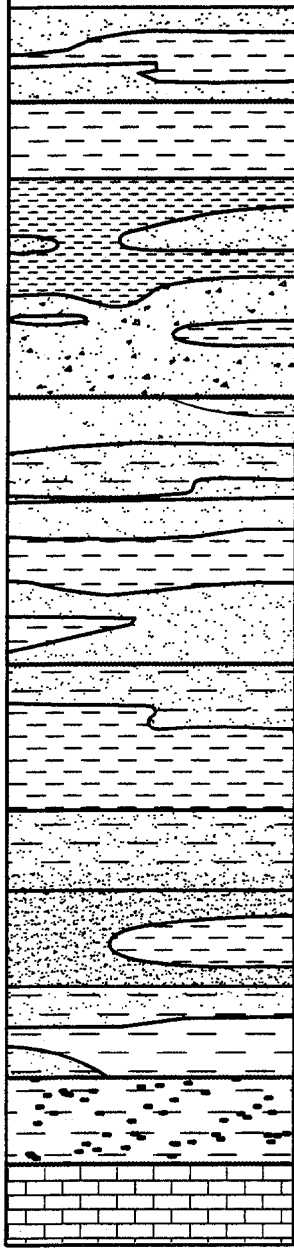
Soil zone 3 consists of an upper sand unit generally present above a depth of 32 ft bgs and often directly underlying the surficial loess deposits. The unit is similar to soil zone 5, consisting of a poorly graded sand, light brown to yellowish brown, grading downward to well graded sand and gravel, light brown to yellowish brown.

Soil zone 2 consists of an upper silt and clay unit directly underlying the surficial loess deposits. The unit is similar to soil zone 4, consisting of a firm, light gray to yellowish brown clayey silt of medium plasticity, with varying sand and gravel content.

Soil zone 1 is the surficial loess. Surficial deposits of loess were encountered from the ground surface to a depth of 15 to 20 ft bgs. The loess deposits consist of a soft to firm, low to moderately plastic silt, light brown or yellowish brown to light gray. The loess is generally late Pleistocene in age. In some areas, there are younger alluvial deposits of Holocene age that fill former erosional features incised in the loess.

3.5 GROUNDWATER HYDROLOGY

The regional groundwater flow systems in the PGDP vicinity occur within the limestone bedrock, McNairy Formation, Terrace Deposits, Lower Continental Deposits, and Upper Continental Deposits. Terms used to describe the hydrogeologic flow system are the Bedrock Aquifer, McNairy Flow System, the Regional Gravel Aquifer (RGA), the UCRS, and the Terrace Deposits Flow System. Specific components of the regional groundwater flow system, shown in Fig. 3.17, have been identified and are defined in the following subsections.

LITHOLOGY	HYDROGEOLOGIC UNITS	FORMATION
	UPPER CONTINENTAL RECHARGE SYSTEM	ALLUVIUM (NOT PRESENT AT SITE 3A)
		LOESS
	REGIONAL GRAVEL AQUIFER	CONTINENTAL DEPOSITS (NOT PRESENT AT SITE 3A)
	TERRACE DEPOSITS FLOW SYSTEM AND EOCENE DEPOSITS	TERRACE DEPOSITS EOCENE FORMATIONS
	PORTERS CREEK CLAY CONFINING UNIT	PORTERS CREEK CLAY
	McNAIRY FLOW SYSTEM	McNAIRY FORMATION
	BEDROCK AQUIFER	RUBBLE ZONE
		MISSISSIPPIAN CARBONATES

SOURCE:
Modified from ORNL 1990

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Figure 3.17. Hydrogeologic units beneath PGDP.

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3.5.1 Bedrock Aquifer

Limestone bedrock subcrops beneath PGDP at depths of 325 to 425 ft bgs. Groundwater production from the bedrock aquifer comes from fissures and fractures within the limestone. Flow direction within the Bedrock Aquifer is expected to approximate that of the overlying McNairy Flow System, flowing to the north and northwest (USGS 1973). Relative to the overlying McNairy Flow System, there is comparatively little flow in the Bedrock Aquifer.

3.5.2 McNairy Flow System

This aquifer consists of the interbedded and interlensing sand, silt, and clay of the Cretaceous McNairy Formation. A weathered rubble zone occurs at the top of the limestone bedrock that is hydraulically connected to the McNairy Flow System. The sand in the McNairy Formation is an excellent aquifer in the southeastern part of the Jackson Purchase Region; however, near PGDP, the McNairy Formation contains significant amounts of silt and clay (LMES 1992). Regionally, the McNairy Formation recharges along areas of outcrop in the eastern part of the region, near Kentucky Lake and Lake Barkley. Water movement is north and northwest toward discharge areas in Missouri and along the Ohio River.

3.5.3 Regional Gravel Aquifer

The RGA underlies the plant area and the area to the north, but pinches out to the south along the slope of the Porters Creek Clay terrace and is not present at Site 3A. The RGA is the dominant groundwater flow system in the area extending from PGDP to the Ohio River. Regional groundwater flow within the RGA trends north-northeast toward a base level represented by the Ohio River. The RGA consists primarily of the coarse sand and gravel facies of the Lower Continental Deposits. Permeable sands of the Upper Continental Deposits and the McNairy Formation, where they occur adjacent to the Lower Continental Deposits, are included in the RGA. Regionally, the RGA includes the Holocene-aged alluvium found adjacent to the Ohio River.

3.5.4 Upper Continental Recharge System

The UCRS consists of a thick, surface loess unit and the Upper Continental Deposits. Groundwater flow in the UCRS is predominantly downward into the RGA, hence the term "recharge system." Vertical hydraulic gradients generally range from 0.5 to 1 ft/ft, where water levels can be measured by wells completed at different depths in the UCRS. In general, the water table is less than 20 ft deep in the western half and south quadrant of PGDP. Depth to water is as much as 40 ft in a broad trough in the water table in the northeast and central areas of PGDP.

3.5.5 Terrace Deposits Flow System

A water table flow system developed in the Terrace Deposits provides some throughflow to the north, across the Porters Creek Clay terrace, ultimately recharging the Upper Continental Deposits beneath the plant. Most of this throughflow is realized east of PGDP, where Pliocene-age gravels are thickest. The water table flow systems immediately south and west of PGDP generally discharge to Bayou Creek or to the adjacent Upper Continental Deposits, because of the shallow depth of the Porters Creek Clay in those areas. Depth to water within the Terrace Deposits south of PGDP ranges from approximately 10 to 35 ft for the time period 1990 through 2000. Reported hydraulic conductivities for these flow systems range from 1E-06 to 1.4E-03 cm/sec (DOE 1996).

At Site 3A, the depth to the water table could not be measured. In soil borings, the use of bentonite drilling mud prevented direct measurement of the water table. In SCPT soundings, pore pressure measurements did not show increasing pressure with depth, so that a water table was not visible in the data. Additionally, equilibrium data from pore pressure dissipation tests did not show the water table either, because of reported negative pore pressure readings or because data did not reach equilibrium conditions. Some of the data from sandier soil zones at depths of 28 to 38 ft bgs suggest the water table is present at depths of 5 to 17 ft bgs. Previous investigations (CH2M Hill 1992) have indicated a water table in the vicinity of Site 3A at depths of 5 to 15 ft bgs, consistent with these results. The inferred flow direction is generally to the northwest, at a horizontal gradient of approximately 0.009 ft/ft, discharging to the UCRS beneath the plant.

The water table fluctuates throughout the year in response to rainfall. At times of the year when the water table is higher, flow within the Terrace Deposits Flow System may discharge locally to tributaries of Bayou and Little Bayou Creeks.

3.6 SUMMARY OF GROUNDWATER HYDROLOGY AT SITE 3A

At Site 3A, water levels were measured indirectly within the Terrace Deposits Flow System during SCPT soundings through measurement of equilibrium pore pressures at various depths. Because of the perched water conditions in several zones, lateral discontinuities, and highly variable fines content, these measurements provide only general information on the direction of groundwater flow and thickness of the more permeable hydrogeologic zones.

The water table appears to be present at depths of 5 to 17 ft bgs. Groundwater is present in thin zones (less than 20 ft thick) that may not be laterally contiguous. The inferred flow direction is predominantly to the northwest, discharging to the UCRS. Lateral flow to incised tributaries of Bayou and Little Bayou Creek is also likely in areas near the creeks, especially during seasonal high water table conditions.

The Porters Creek Clay, which is at least 90 ft thick at Site 3A, effectively separates the Terrace Deposits Flow System from underlying aquifers. The McNairy Flow System and underlying Bedrock Aquifer are, therefore, not of significant hydrogeologic interest at Site 3A. The RGA and UCRS are not present at Site 3A.

3.7 GEOTECHNICAL SOIL PROPERTIES AT SITE 3A

Geotechnical soil properties measured at Site 3A include both field and laboratory measurements of soil type, index characterization, contaminant transport, consolidation, strength, and permeability.

3.7.1 Standard Penetration Test Results

Standard penetration testing involves driving a split spoon sampler into the bottom of a borehole using a hammer of standard configuration. By counting the number of blows of the hammer required to drive the sampler 12 inches, a measurement of the sample's relative density or hardness can be made. This number of blows is referred to as the N-value for that sample depth. At Site 3A, field measurements of SPT N-values were obtained from both the shallow and deep mud rotary boreholes. The results of standard penetration testing can usually be correlated in a general way with the pertinent physical properties of the soil. The correlation for clays can be regarded as no more than an approximation, but that for sands is often reliable enough to permit the use of N-values in foundation design (Peck, Hanson, and Thornburn 1974).

Figure 3.18 summarizes the SPT N-values obtained from all borings and from depths up to 70 ft bgs. The data show wide variability, which is common in heterogeneous soil zones such as those present at Site 3A. For this reason, the soil profile has been delineated into six different soil zones, representing generally different soil types or depths of deposition, as discussed in Sect. 3.3. Table 3.1 lists the six soil zones that were delineated for purposes of this analysis and their respective average N-values.

Table 3.1. Standard Penetration Test results for various soil zones

		Values from SPT borings				Values from SCPT soundings	
Soil zone	Soil description	Average N-value (blows/ft)	Standard deviation N-value (blows/ft)	Average N'_{60}-value (blows/ft)	Standard deviation N'_{60}-value (blows/ft)	Average N'_{60}-value (blows/ft)	Standard deviation N'_{60}-value (blows/ft)
1	Loess, soft to firm, low to moderately plastic silt	11.7	8.0	18.1	16.7	12.8	11.4
2	Terrace Deposits, firm, medium plasticity clayey silt	13.0	3.5	11.8	3.1	23.5	13.2
3	Terrace Deposits, dense, fine, poorly graded sand/gravel	32.6	22.7	28.7	20.2	48.0	20.9
4	Terrace Deposits, firm, medium plasticity silt/clay	16.5	11.9	12.7	9.4	16.6	8.5
5	Terrace Deposits, dense, fine to medium, poorly graded sand/gravel	33.5	26.8	22.7	17.8	37.3	23.4
6	Porters Creek Clay, firm to hard, low plasticity laminated silt/clay	48.2	23.4	31.6	15.5	31.9	7.2

Figure 3.19 summarizes the SPT N-values obtained from all borings up to 70 ft bgs, but differentiated based on the soil zone designated for the specific sample. These plots do not show any specific trends with depth but clearly show differences between soil zones. For soil zones 1 and 2, which consist predominantly of medium to stiff silts and clays, SPT N-values generally range from 5 to 15 blows/ft. For soil zone 4, which also consists of silts and clays, a broader range of SPT N-values was measured, predominantly between 5 and 25 blows/ft. For soil zone 6, representing the harder silts and clays of the Porters Creek Clay, the data show broad scatter, but are generally over 30 blows/ft. Data are broadly scattered for soil zones 3 and 5, which represent loose to very dense sand and gravel deposits.

In saturated, fine or silty, dense or very dense sand, the N-values may be abnormally high because of the tendency of such materials to dilate during shear under undrained conditions. In addition, the N-value in sands is influenced to some extent by the depth at with the test is made, because of the greater confinement caused by increasing overburden pressure. For this reason, a correction factor (C_N) is applied to the field N-values in sands to obtain a corrected N-value that corresponds to an effective overburden pressure of 1 ton per square foot (tsf). This correction factor (ASTM D6066) is approximated by

$$C_N = (1/p')^{0.5}$$

where

$0.5 < C_N < 2.0$, and

p' = effective overburden pressure in tsf.

In addition, N-values can be corrected for the rod energy delivered to the SPT sampler. A Foremost Mobile SPT Automatic Hammer was used in all mud rotary boreholes. The hammer has a calibrated overall average SPT energy factor of 96.4%, which was applied as a further correction. The resulting corrected N-values are referred to as N'_{60}-values.

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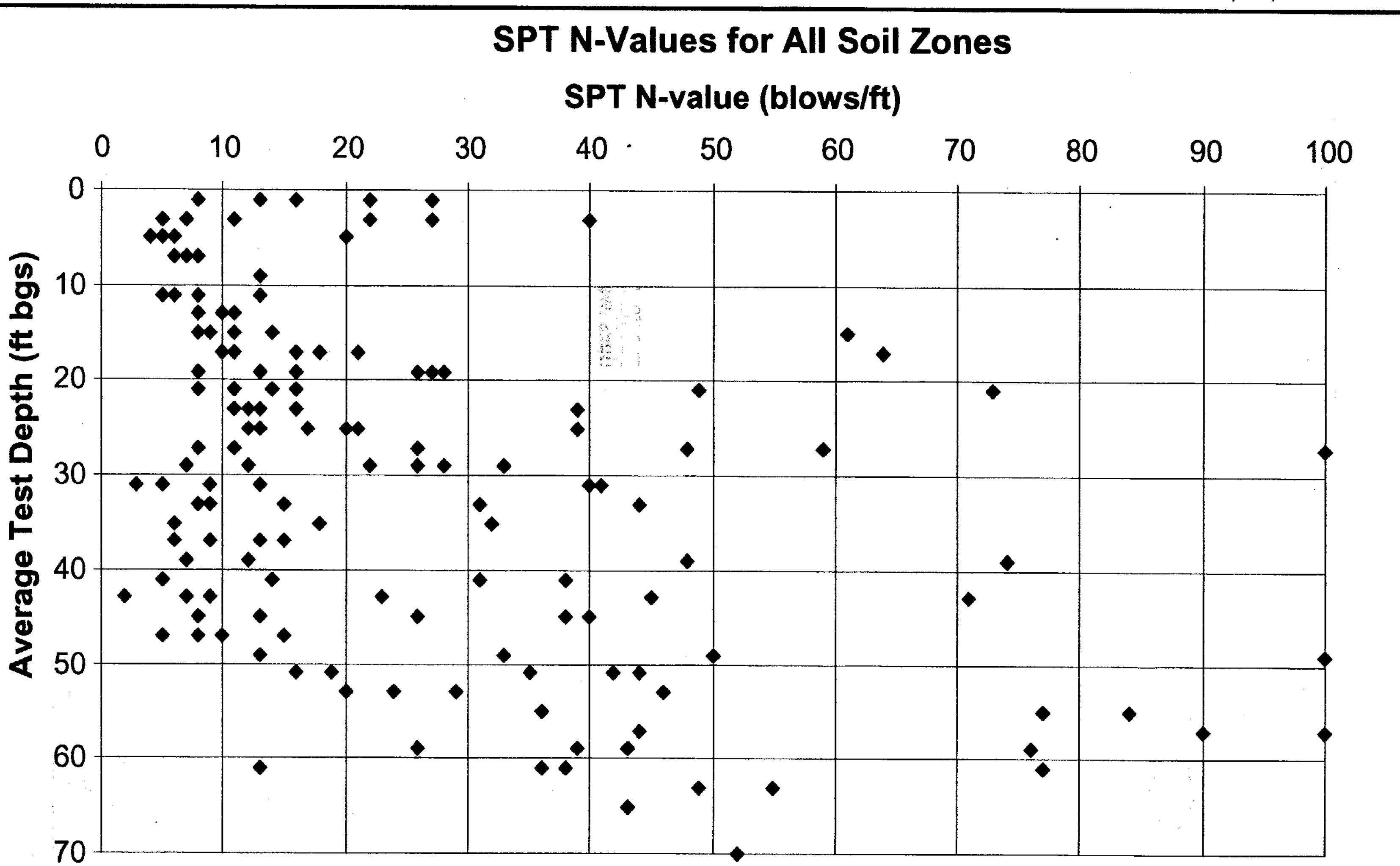

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Fig. 3.18. Plot of N-value vs. depth.

Figure No. /99049/DWGS/P34NVAL
DATE 07-26-02

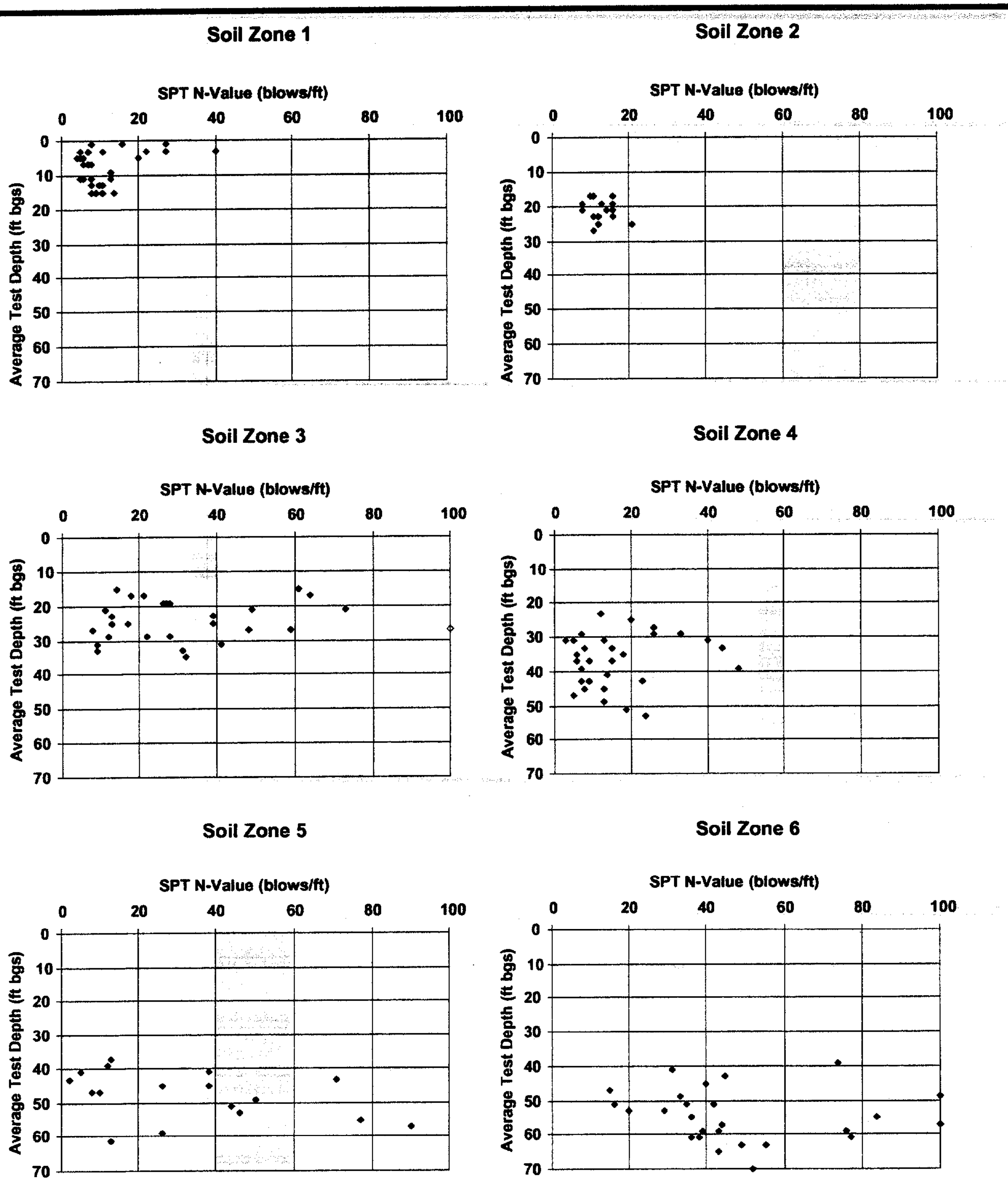

Fig. 3.19. Plots of N-value vs. depth for different soil zones.

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Figure 3.20 shows the corrected N'_{60}-values versus depth for all soil zones. Similar to Fig. 3.19, these plots do not show any specific trends with depth, but clearly show differences between soil zones. Table 3.1 lists the average of the corrected N'_{60}-values. Correcting the N-values for effective overburden pressure results in a reduction in the average N_{60}-values for soils below 16 ft depth. The impacts of this reduction on liquefaction potential, particularly for the sandy soils in soil zones 3 and 5, are discussed in Chap. 5.

Cone penetrometer tip resistance has been related to SPT N-values so that SCPT data can be used in existing SPT-based design approaches. Numerous factors affect the SCPT/SPT correlation, including the rod energy delivered to the SPT sampler, the depth of the sample, the fines content and grain size of the sample, and the specific soil behavior type (Lunne, Robertson, and Powell 1997). The SCPT data presented in Appendix E have been correlated to equivalent N-values corresponding to an average energy ratio of 60% and corrected for depth. These values, referred to as N'_{60}-values, are shown on the SCPT profiles in Appendix E. Table 3.1 lists the average corrected N'_{60}-values as interpreted from the SCPT data. In general, the corrected N'_{60}-values derived from the SCPT data show reasonable correlation to the measured N'_{60}-values from the SPT borings.

Figures 3.21 and 3.22 present profiles of corrected N'_{60}-values derived for paired sets of SPT borings and SCPT soundings (i.e., borings and soundings drilled near one another). These plots show relatively good correlation between the borings and soundings with respect to corrected N'_{60}-values. The following summarizes the comparisons of corrected N'_{60} plots between paired sets of borings and soundings.

- Boring SB01 and sounding SC01 were installed in the southwest corner of Site 3A; a dense sand layer at 20 to 21 ft bgs is apparent in both profile plots, although sounding SC01 encountered a dense deposit between 17 to 19 ft bgs as well.

- Boring SB02 and sounding SC04 were installed in the northwest corner of Site 3A; a dense sand layer at 20 to 22 ft bgs is apparent in both profiles. However, in sounding SC04, a second very dense sand and gravel deposit was also encountered between 24 to 29 ft bgs as well. Below 40 ft bgs, interbedded sands and gravels of varying density were encountered in both the boring and sounding, but the depths of the denser zones do not match between them.

- Boring SB03 and sounding SC09 were installed in the center of Site 3A nearly 60 ft apart. A very dense sand deposit was encountered in SB03 at 24 to 28 ft bgs that was evident in SC09 at 30 to 34 ft bgs. Similarly, a dense gravel deposit was encountered in SB03 just above the top of the Porters Creek Clay at 48 ft bgs; dense sands and gravels were also encountered in SC09 between 46 and 49 ft bgs.

- Boring DB02 and sounding SC08 were installed in the southeast portion of the site near Hobbs Road and are located nearly 126 ft apart. A moderately dense sand was encountered in both at a depth of 29 ft bgs. A very dense sand and gravel deposit was encountered in DB02 between 54 and 58 ft bgs; this same deposit was encountered in SC08; however, the materials were very dense over a broader depth range of 50 to 60 ft bgs.

In summary, results of standard penetration testing at Site 3A (as defined by the N'_{60}-values) show that the clayey and silty soil zones (1, 2, and 4) are medium stiff to stiff, with only slight increases in N'_{60}-values with depth. Data for the sandier soil zones (3 and 5) are broadly scattered, which indicates a range of loose to very dense deposits. N'_{60}-values for soil zone 6, representing the Porters Creek Clay, also show broad scatter but average over 30 blows/ft, which indicates firm to hard deposits. Plots of the N'_{60}-values show relatively good correlation between paired sets of SPT borings and SCPT soundings.

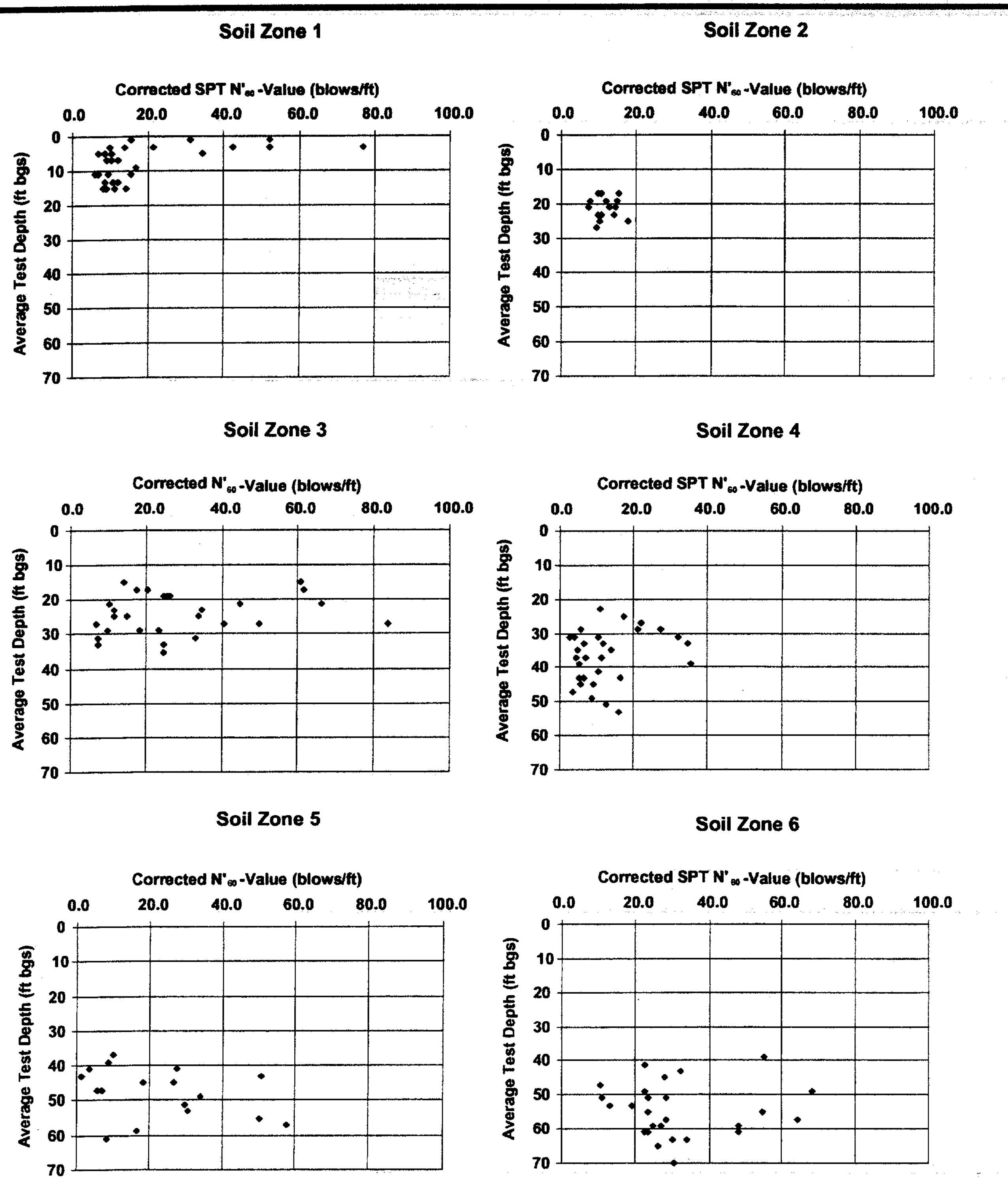

Fig. 3.20. Plots of corrected N'_{60}-value vs. depth for different soil zones.

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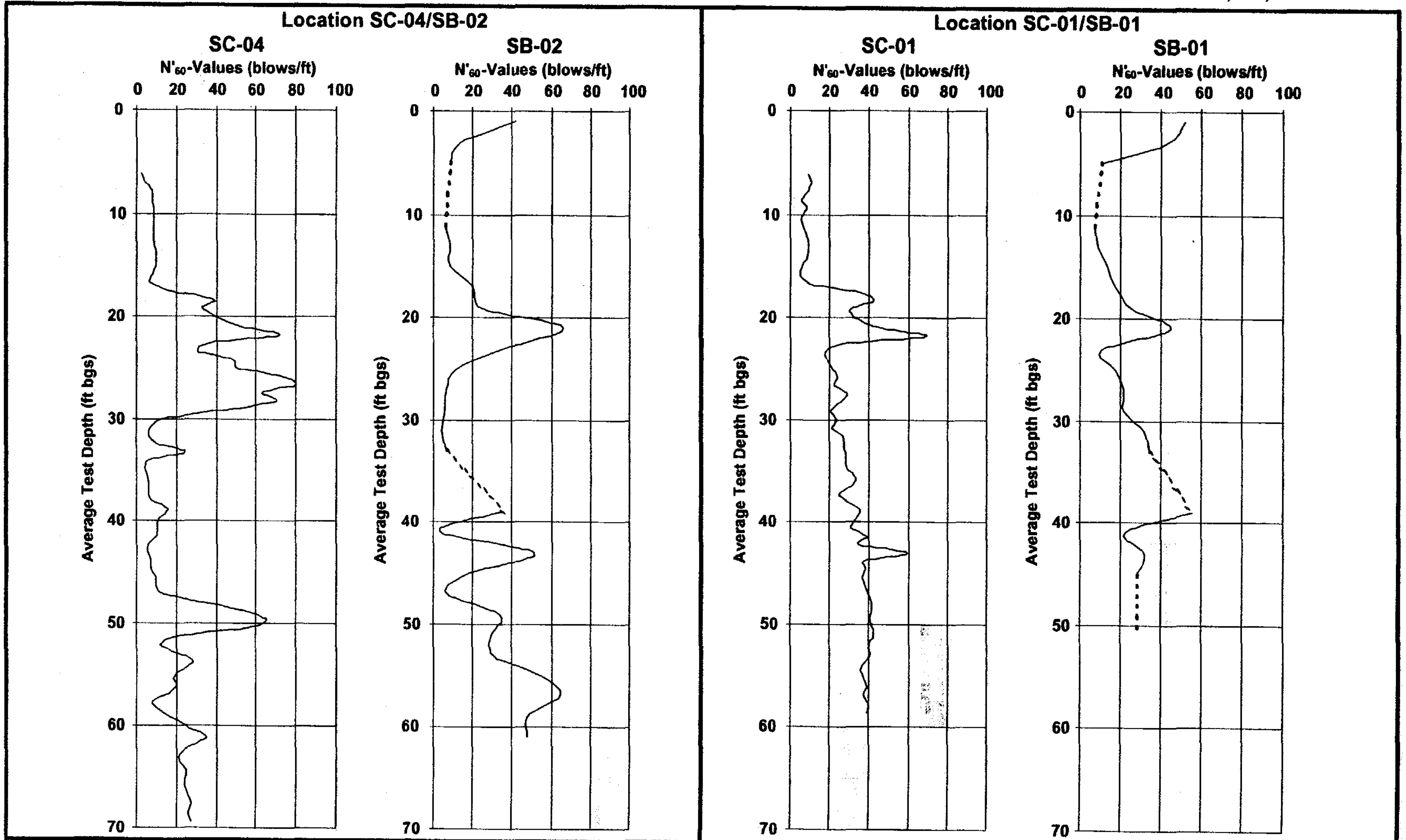

LEGEND

———— ....Measured values

– – – – ....Interpolated values

NOTES:

1. SC-04 and SB-02 are located approximately 6 ft apart
2. SC-01 and SB-01 are located approximately 1 ft apart

Fig. 3.21a. Plots comparing N'_{60}-value vs. depth for paired borings and soundings.

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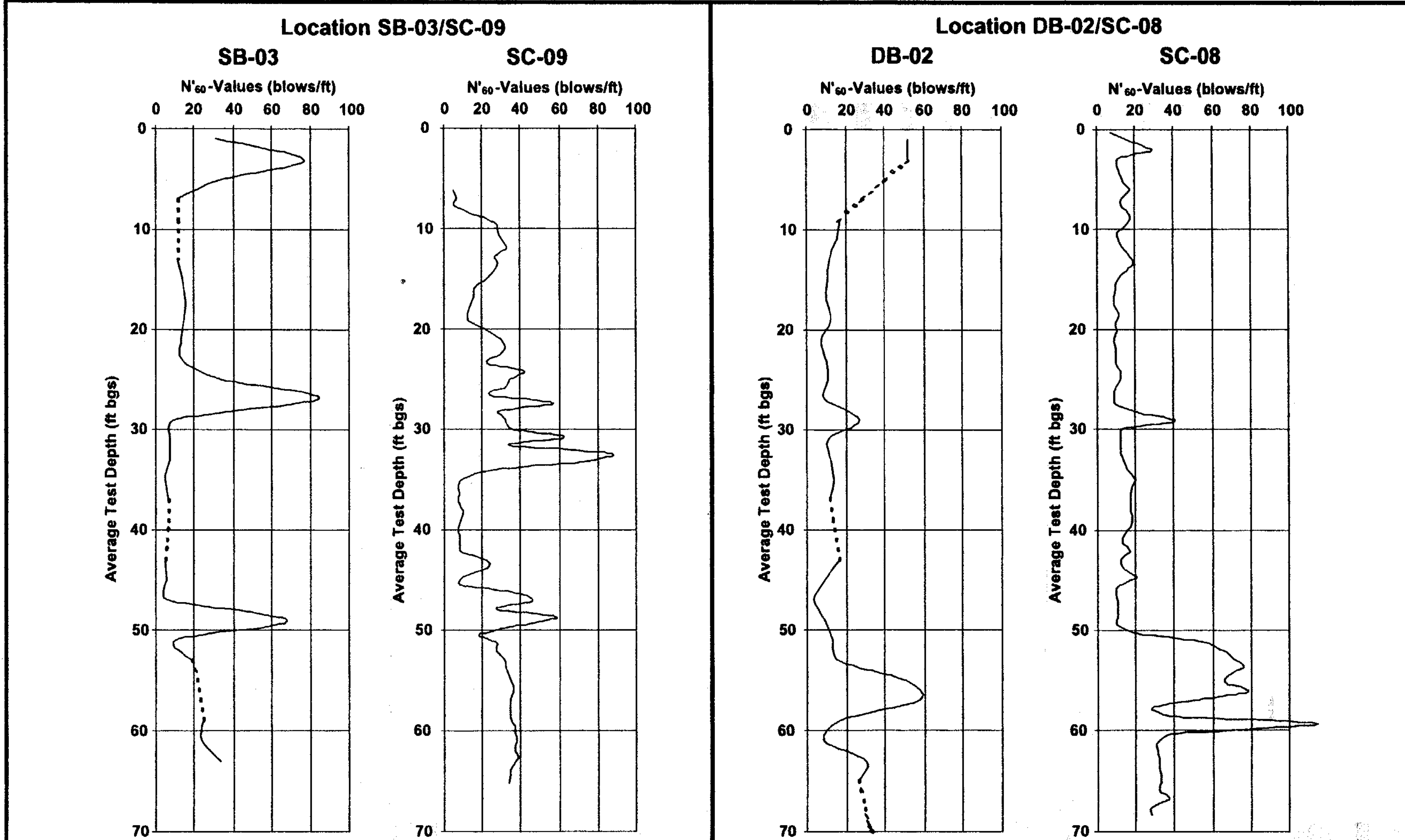

LEGEND

——— ....Measured values

– – – – ....Interpolated values

NOTES:

1. SB-03 and SC-09 are located approximately 60 ft apart
2. DB-02 and SC-08 are located approximately 126 ft apart

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Fig. 3.21b. Plots comparing N'$_{60}$-value vs. depth for paired borings and soundings.

Figure No. /99049/DWGS/P34NBOR2

DATE 07-29-02

3.7.2 Geotechnical Laboratory Test Results

The purpose of the geotechnical laboratory testing is to collect information on site-specific soil behavior for use in subsequent analysis of liquefaction, fate and transport, settlement, bearing capacity, and seismic design at Site 3A. Table 3.2 lists the types of geotechnical properties that were tested, the purpose of each test, and where the test results are subsequently used in this report.

Index properties are tested so as to characterize the type of soil in a sample and are subsequently used in the liquefaction, settlement, and bearing capacity models in Chaps. 4 and 5. Fate and transport properties are tested to assess the contaminant release and migration behavior of a soil sample and are used in the fate and transport model in Chap. 4. Consolidation properties are tested to assess the amount and rate of consolidation of a sample and are used in the settlement model in Chap. 4. Strength properties are tested to assess the bearing and shear strength of a soil sample for use in the bearing capacity model in Chap. 4. Seismic properties are tested to assess the peak ground acceleration and design ground motions in the seismic design model in Chap. 7.

Geotechnical laboratory analyses were conducted on 48 split spoon samples and 44 Shelby tube samples taken from the soil borings. Results of the geotechnical analyses are summarized in Tables 3.3 and 3.4. The geotechnical laboratory data sheets are included in Appendix E. The following discusses the geotechnical soil properties for each of the six soil zones.

3.7.2.1 Index Properties

Geotechnical index properties are tests used to characterize the type of soil in a sample by describing its wetness, plasticity, unit weight, proportion of soil voids, and proportion of sand, silt, or clay-sized particles. These results are subsequently used to support analysis of liquefaction, settlement, and bearing capacity in Chaps. 4 and 5.

Geotechnical index properties of soils include moisture content, Atterberg limits, specific gravity, density, void ratio, and grain size distribution. Moisture content, Atterberg limits, specific gravity, and grain size distribution were measured in disturbed split spoon samples; moisture content, density, and void ratio were measured in relatively undisturbed Shelby tube samples. Table 3.5 presents the average values of the geotechnical index properties as measured for each soil zone.

Moisture content. Natural moisture content measures the percentage of the weight of the water within the soil to the dry weight of the soil aggregate, and is, therefore, indicative of the degree of wetness of the soil. The moisture contents of the split spoon samples varied between 11.2 and 67.6%. Lower moisture contents were typically measured for samples in the sandier zones, with an average of 15.1% in zone 3 and 19.2% in soil zone 5. Higher moisture contents were typically measured in the clays and silts, with an average of 34.7% in zone 4 and 55.9% in zone 6. In soil zone 6, the moisture content was frequently lower than the sample's plastic limit, which would be consistent with an overconsolidated soil deposit.

The moisture contents of the Shelby tube samples showed good correlation with split spoon samples. Moisture contents varied between 13.8 and 75.3%, with higher moisture content (average 59.4%) measured in zone 6 soil samples.

Atterberg limits. Atterberg limits tests were conducted on split spoon samples, and are indicative of the plasticity of the fine-grained fraction of the soil. The liquid limit represents the moisture content of the soil at a point where the soil begins to "flow," and is the upper (wetter) limit of the soil's plastic range. The plastic limit represents the moisture content of the soil at a point where the soil begins to "crumble," and is the lower (drier) limit of the soil's plastic range.

Table 3.2. Explanation of geotechnical testing

Geotechnical property	Why tested	Where used
Index properties	To characterize the type of soil in a sample by defining the following:	
Moisture content	• degree of wetness of soil	Liquefaction potential, Sect. 5.6
Atterberg limits	• plasticity of fine-grained soil	Liquefaction potential, Sect. 5.6
Specific gravity	• weight of soil grains	Weight-volume calculations
Dry density	• weight of soil mass	Settlement model, Sect. 4.3; Bearing capacity model, Sect. 4.4
Void ratio	• proportion of soil voids	Settlement model Sect. 4.3
Grain size distribution	• proportion of sand, silt, and clay	Liquefaction potential, Sect. 5.6
Fate and transport properties	To assess contaminant release and migration behavior by defining the following:	
Hydraulic conductivity (Shelby tube)	• vertical permeability of fine-grained soil	Fate and transport model, Sect. 5.2
Hydraulic conductivity (pore pressure dissipation)	• horizontal permeability of coarse-grained soil layer	Fate and transport model, Sect. 5.2
Partitioning coefficient	• contaminant affinity to soil mass	Fate and transport model, Sect. 5.2
Consolidation properties	To assess the amount and rate of consolidation by defining the following:	
Preconsolidation pressure	• maximum historical soil load	Settlement model, Sect. 4.3
Compression index	• load-settlement relationship	Settlement model, Sect. 4.3
Coefficient of consolidation	• time rate of consolidation	Settlement model, Sect. 4.3
Strength properties	To assess the bearing and shear strength of soil by defining the following	
U-U shear strength	• frictional and cohesional strength behavior	Bearing capacity model, Sect. 4.4
Seismic properties	To assess the PGA and design ground motion by defining the following:	
Shear wave velocity	• seismic shear-wave velocity profile	Seismic design model, Sect. 7.2

U-U = unconsolidated-undrained

Table 3.3. Results of geotechnical testing of split spoon samples

Station	Sample no.	Depth (feet)	USCS symbol	Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Gravel (%)	Sand (%)	Fines (%)	Specific gravity	Partitioning coefficient (L/kg)	
												^{237}Np	^{99}Tc
SB-01	3	4-6	CL	27.6	31	19	12	0.1	2.0	97.9	2.68	NA	NA
	4	10-12	CL	22.9	28	17	11	0.4	12.2	87.4	2.63	38	NR
	8	18-20	SC	13.3	21	12	9	15.3	47.6	37.1	2.67	NA	NA
	11	24-26	MH	64.4	102	53	50	0.0	6.5	93.5	2.58	NA	NA
	15	32-34	MH	55.4	95	53	42	0.0	29.4	70.6	2.65	NA	NA
	19	44-46	MH	66.4	103	60	43	0.0	25.4	67.9	2.56	NA	NA
SB-02	3	4-6	CL	24.6	32	20	12	0.4	1.9	97.7	2.68	NA	NA
	5	12-14	CL	23.3	30	19	11	0.0	4.1	95.9	2.68	NA	NA
	9	20-22	SC	14.6	29	16	13	22.0	58.6	19.4	2.62	402	0.17
	15	32-34	CL-ML	17.9	20	13	7	0.6	43.0	56.3	2.67	NA	NA
	19	44-46	SP-SM	21.6	NP	NP	NP	18.5	72.5	8.9	2.67	NA	NA
	22	50-52	SP-SM	22.8	NP	NP	NP	0.0	87.6	12.4	2.67	NA	NA
	24	54-56	SM	56.9	81	60	21	3.0	74.3	25.7	2.58	NA	NA
	27	60-62	SM	67.6	90	65	26	0.0	78.0	22.0	2.60	NA	NA
SB-03	4	6-8	CL	23.7	31	20	11	0.1	2.9	97.1	2.70	NA	NA
	7	16-18	CL	20.9	30	16	14	0.3	8.2	91.5	2.47	NA	NA
	11	24-26	CL	14.3	21	12	9	4.0	39.4	55.5	2.66	48	NR
	17	36-38	CL	30.0	43	21	22	0.0	8.0	92.0	2.55	NA	NA
	19	44-46	MH	62.6	106	50	56	0.0	12.6	87.4	2.25	NA	NA
	23	52-54	CL	21.3	28	15	13	1.5	37.1	61.4	2.38	NA	NA
	26	62-64	SM	56.9	91	65	26	0.0	77.3	22.7	2.04	NA	NA
SB-05	4	6-8	CL	26.9	38	20	18	2.0	4.6	93.4	2.64	305	0.16
	6	14-16	ML	17.0	NP	NP	NP	0.0	2.4	97.6	2.07	NA	NA
	10	22-24	GC	15.0	NA	NA	NA	30.2	28.7	41.1	2.36	NA	NA
	14	30-32	SC	23.8	39	14	25	15.5	58.2	26.3	2.46	NA	NA
	18	42-44	SC	23.7	46	15	31	21.6	33.7	44.6	NA	NA	NA
	19	48-50	MH	66.2	103	61	42	0.0	30.4	69.6	2.24	NA	NA
	21	52-54	SM	62.3	89	65	24	0.0	84.7	15.3	2.23	NA	NA
SB-06	3	6-8	CL	29.2	35	20	15	0.0	2.3	97.7	2.65	30	0.13
	6	14-16	CL	22.0	32	16	16	2.1	9.9	88.0	2.64	NA	NA
	10	22-24	CL	13.3	28	13	15	7.4	32.5	60.1	2.65	NA	NA
	14	30-32	GW-GC	16.3	28	16	12	54.3	38.0	7.7	2.61	NA	NA
	18	38-40	SC	17.8	27	14	13	21.4	39.1	39.5	2.64	NA	NA
	20	42-44	SP	11.2	NA	NA	NA	0.0	98.8	1.2	NA	NA	NA
	22	46-48	MH	66.7	117	50	67	0.0	4.1	95.9	2.50	232	NR
	23	54-56	SM	18.8	85	63	22	0.0	76.6	23.4	2.50	NA	NA

Table 3.3. Results of geotechnical testing of split spoon samples (continued)

Station	Sample no.	Depth (feet)	USCS symbol	Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Gravel (%)	Sand (%)	Fines (%)	Specific gravity	Partitioning coefficient (L/kg)	
												^{237}Np	^{99}Tc
DB-02	3	8-10	CL	18.7	29	20	9	0.5	4.6	94.9	2.62	NA	NA
	7	16-18	CL	24.3	40	16	24	0.1	6.8	93.1	2.63	NA	NA
	11	24-26	CL	21.7	32	14	18	2.4	32.1	65.6	2.63	NA	NA
	14	30-32	CL	19.9	25	12	13	5.0	37.0	58.0	2.63	NA	NA
	17	36-38	CL	24.9	39	18	21	0.2	15.1	84.7	2.70	153	0.2
	19	44-46	SC	21.5	29	13	16	7.1	63.3	29.6	2.60	NA	NA
	21	48-50	ML	24.7	44	NP	NP	0.1	11.5	85.4	2.66	47	NR
	24	54-56	SM	18.7	NP	NP	NP	8.6	61.3	30.1	2.64	NA	NA
	27	60-62	SM	21.0	NA	NA	NA	21.3	58.2	20.5	2.78	116	0.32
	30	69-70	SM	61.4	90	82	8	0.0	68.3	31.7	2.53	14	1.42
	31	89-91	SM	66.7	97	70	27	0.5	71.7	27.8	2.52	79	0.53
	32	109-111	SM	60.0	90	63	27	0.0	75.7	24.3	2.51	NA	NA

CL = lean clay
GC = clayey gravel
GW = well-graded gravel
MH = elastic silt
ML = inelastic silt
NA = not analyzed
NP = non-plastic
NR = not reported (no breakthrough of permeant)
SC = clayey sand
SM = silty sand
SP = poorly graded sand
USCS = Unified Soil Classification System (ASTM D 2487)

Table 3.4. Results of geotechnical testing of Shelby tube samples

Station	Sample no.	Depth (ft)	Moisture content (%)	Dry density (pcf)	Permeability (cm/sec)	Initial void ratio, e_0	Hypothetical preconsolidation pressure, σ'_p, (tsf)	Virgin compression index, C_c	Coefficient of consolidation, c_v (cm^2/sec)	Apparent friction angle, φ (degrees)	Apparent cohesion, c (tsf)
SB-01	1	6-8	28.5	98.8	1.4E-07	0.671	0.57	0.12	0.007	5.9	0.42
	2	8-10	20.4	107.1	4.9E-07	0.561	0.74	0.11	0.007	1.7	1.33
	3	34-34.6	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	4	36	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	5	46-47.5	57.3	60.2	3.3E-05	1.986	7.35	0.44	0.122	N/A	N/A
	6	48-49.5	59.6	59.6	3.0E-05	1.956	7.34	0.41	0.138	N/A	N/A
SB-02	1	6-7.9	28.6	96.6	3.3E-07	0.897	2.36	0.24	0.028	2.6	0.56
	2	8-9.6	22.5	104.0	2.7E-07	0.627	0.93	0.11	0.075	10.8	1.81
	3	34-36	17.8	113.6	2.9E-07	0.438	3.97	0.12	0.022	10.8	0.89
	4	36-38	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	5	62-63.5	63.2	57.8	5.0E-05	1.899	7.32	0.34	0.139	N/A	N/A
	6	64-65.2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
SB-03	1	8-9.4	23.0	105.0	1.4E-06	0.679	1.02	0.13	0.065	0.0*	7.14*
	2	10-11.9	20.9	72.8	3.2E-07	0.593	4.39	0.10	0.052	16.3	2.06
	3	38-40	23.0	103.0	6.6E-08	0.616	4.10	0.17	0.033	3.3	2.63
	4	40-42	25.9	98.7	4.4E-08	0.685	5.14	0.24	0.011	3.5	2.20
	5	54	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	6	56-57.6	59.8	57.2	1.1E-05	1.989	4.19	0.45	0.073	N/A	N/A
	7	64-65	59.6	60.3	N/A	2.025	3.10	0.39	0.061	N/A	N/A
	8	66-67	56.1	59.7	1.5E-05	2.114	2.1	0.32	0.076	N/A	N/A
SB-05	1	8-10	22.8	103.4	1.4E-07	0.661	0.33	0.063	.024	12.1	0.76
	2	10-11.5	23.6	104.1	1.5E-07	0.7389	1.65	0.13	0.023	8.5	0.21
	3	32-34	13.8	119.8	3.2E-08	0.440	4.85	0.10	0.046	5.3	0.73
	4	34-36	15.8	116.2	7.0E-08	0.521	1.89	0.12	0.066	4.5	1.55
	5	44-45.8	60.3	59.1	5.4E-08	2.020	8.00	0.48	0.069	N/A	N/A
	6	46-47.6	75.3	51.0	N/A	2.044	3.43	0.42	0.061	N/A	N/A
	7	54-55.3	59.1	59.1	1.8E-07	1.823	3.17	0.30	0.057	N/A	N/A
	8	56-57.3	60.8	59.3	1.8E-05	1.857	7.42	0.27	0.082	N/A	N/A
SB-06	1	4-5.9	26.8	99.2	9.3E-06	0.713	0.80	0.095	.065	15.8	0.0
	2	8-10	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	3	48-49.5	58.4	63.1	5.4E-08	2.015	4.11	0.33	.130	N/A	N/A
	4	50	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	5	52-53	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
DB-02	1	4-6	25.6	116.8	8.4E-06	0.750	1.05	0.105	0.061	11.9	1.36
	2	6-8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	3	38-39.7	21.4	106.5	2.0E-06	0.617	7.3	0.17	0.103	10.4	1.2
	4	71-71.8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	5	73-73.9	61.5	57.4	4.2E-05	2.035	6.51	0.43	0.148	N/A	N/A

Table 3.4. Results of geotechnical testing of Shelby tube samples (continued)

Station	Sample no.	Depth (ft)	Moisture content (%)	Dry density (pcf)	Permeability (cm/sec)	Initial void ratio, e_0	Hypothetical preconsolidation pressure, σ'_p, (tsf)	Virgin compression index, C_c	Coefficient of consolidation, c_v (cm^2/sec)	Apparent friction angle, φ (degrees)	Apparent cohesion, c (tsf)
DB-02	6	93-95	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
(cont.)	7	95-96.2	57.8	57.6	4.9E-07	2.045	4.88	0.53	0.053	N/A	N/A
	8	111-112.5	54.6	63.8	N/A	1.596	7.50	0.29	0.056	N/A	N/A
	9	113-113.9	57.8	61.9	1.4E-06	1.783	8.00	0.30	0.029	N/A	N/A
	10	131-131.8	57.8	64.2	1.9E-07	1.557	6.50	0.09	0.020	N/A	N/A
	11	133-133.9	50.8	65.5	3.2E-05	1.424	3.55	0.27	0.025	N/A	N/A

*U-U test conducted at one confining pressure only so that φ and c values may not reflect the true slope of the rupture envelope.
NA = not analyzed
pcf = pounds per cubic foot

Table 3.5. Average index properties for each soil zone

Index property	units	Soil zone 1	2	3	4	5	6
Moisture content, w, (split spoon)	%	24.3	20.1	15.1	34.7	19.2	55.9
Moisture content, w, (Shelby tube)	%	24.3	—	—	19.6	—	59.4
Liquid limit, LL	%	32	32	25	56	28	89
Plastic limit, PL	%	19	15	14	28	14	60
Specific gravity, $S.G.$	—	2.66	2.60	2.50	2.57	2.67	2.43
Dry density, γ_d	pcf	100.8	—	—	109.6	—	59.8
Initial void ratio, e_0	—	0.69	—	—	0.55	—	1.89
Fines content, $P200$, (< 0.075 mm)	%	94.4	77.6	43.1	69.9	20.3	40.6

— indicates no Shelby tube samples tested from this soil zone.
pcf = pounds per cubic feet

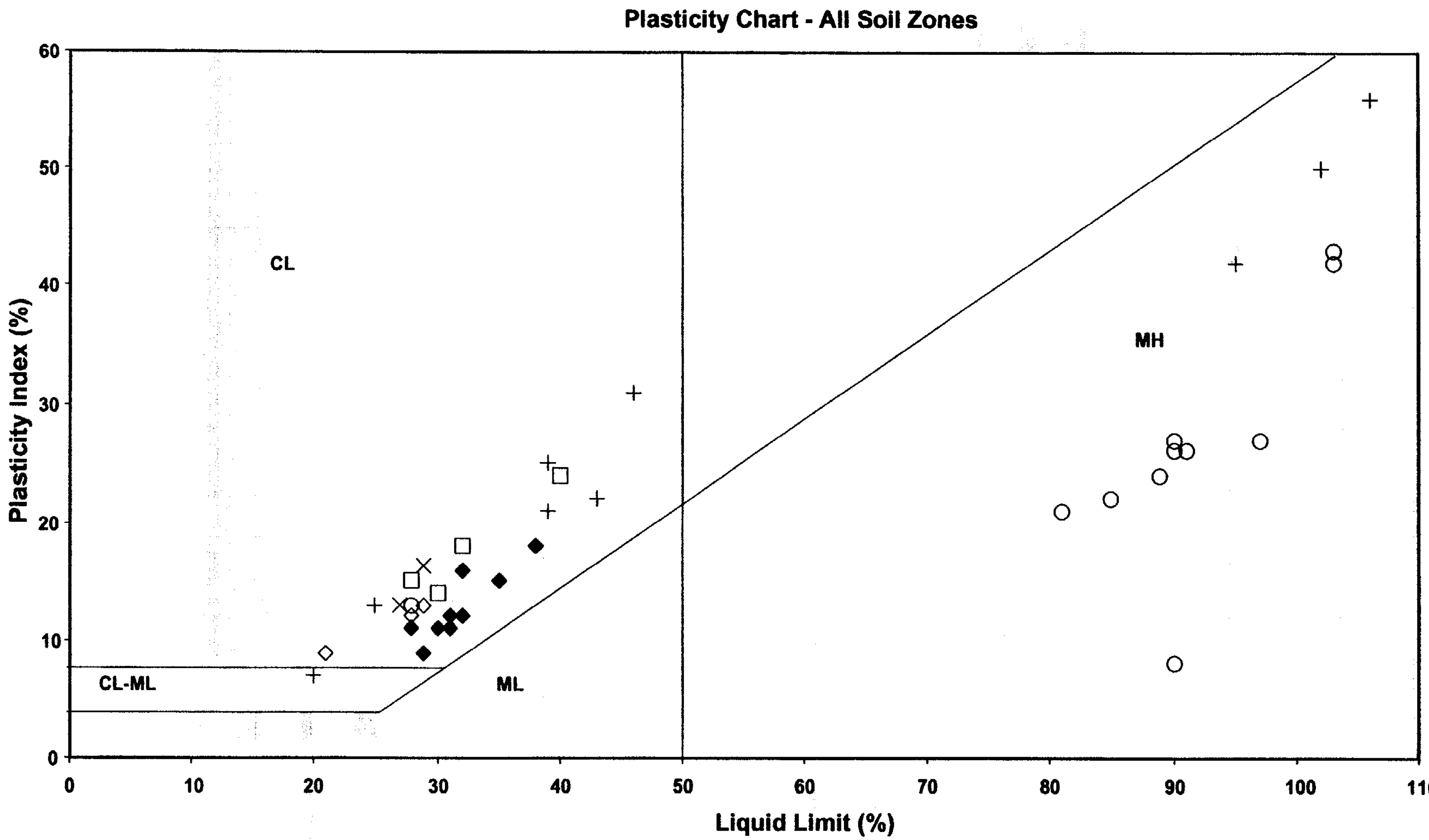
Soils in zones 1 and 2 consist predominantly of moderately plastic lean clays with liquid limits ranging from 28 to 40, and averaging 32. Zone 4 soils consist of lean clays similar to zone 2 soils, but three samples consisted of higher plasticity elastic silts with liquid limits as high as 106. The average liquid limit for zone 4 soils is 56. Liquid limits for soils in zones 3 and 5 represent the plasticity of the fine-grained fraction of the sample; these are typically lower plasticity silts and clays, with liquid limits ranging from 21 to 29, with some non-plastic soil fractions. Liquid limits for soils in zone 6, the Porters Creek Clay, are generally between 81 and 117, typical of very high plasticity elastic silts, although one sample from soil zone 6 had a liquid limit of only 28, and may represent a transition from zone 4 soils.

Plastic limits test results reveal trends similar to the liquid limits test results, with lower values measured for soils in zones 3 and 5 and higher values measured in soil zone 6. Plasticity charts, which are plots of liquid limit versus plastic limit, can be used during design to correlate (indirectly) the properties of silts and clays, such as their dry strength, compressibility, reaction to shaking, and consistency. A plasticity chart is presented in Fig. 3.23 for all samples from the six soil zones. The chart indicates the Unified Soil Classification System (USCS) group symbols for the fine-grained fractions of each soil sample.

Specific gravity, dry density, and void ratio. Specific gravity, dry density, and void ratio are index properties used to define the weight-volume relationships of soils and are indicative of the degree of compactness of the soil, as well as the relative weight of the soil aggregate. Based on the average values presented in Table 3.5, soils in zones 1 through 5 exhibit similar characteristics, with specific gravities averaging between 2.50 and 2.67. Unit weight and void ratio could only be measured in relatively undisturbed Shelby tube samples, which were not taken from soils in zones 2, 3, or 5. Dry densities for soils in zones 1 and 4 averaged 100.8 and 109.6 pounds per cubic foot (pcf), respectively; void ratios for zones 1 and 4 soils averaged 0.69 and 0.55, respectively. These results indicate that soils in these zones are loose to moderately compact.

Results for soils in zone 6, the Porters Creek Clay, are notably different from those of the overlying deposits. The specific gravity is lower, averaging 2.40, indicating the presence of lighter-weight soil constituents or minerals. The dry density of soils in zone 6 averages only 59.8 pcf and the void ratio is high, averaging 1.89, which indicates that the soil is light weight and loose.

Grain size distribution. Grain size distribution is used to describe the soil composition and classification. The grain size distribution is quite variable in soils at Site 3A, even within individual soil zones. Therefore, this discussion of grain size is a generalization of overall conditions and is not to be considered indicative of individual samples or lenses within a given soil zone.



LEGEND

◆.....................Soil zone 1
□.....................Soil zone 2
◇.....................Soil zone 3
+.....................Soil zone 4
×.....................Soil zone 5
○.....................Soil zone 6

CL ....Lean clay
CH........................Fat clay
ML...............................Silt
MH.................... Elastic silt

SOURCE:

Unified Soil Classification System (ASTM D2487)

Not to scale

U.S. DEPARTMENT OF ENERGY
DOE OAK RIDGE OPERATIONS
PADUCAH GASEOUS DIFFUSION PLANT

BECHTEL JACOBS COMPANY, LLC
MANAGED FOR THE US DEPARTMENT OF ENERGY UNDER US GOVERNMENT CONTRACT DE-AC-05-98OR22700
Oak Ridge, Tennessee • Paducah, Kentucky • Portsmouth, Ohio

SAIC
Science Applications International Corporation
P.O. Box 2502
Oak Ridge, Tennessee 37831

Fig. 3.22. Plasticity chart for all soil zones.

Figure No. /99049/DWGS/P34PLST2

DATE 07-26-02

Soils in zone 1 exhibit the greatest uniformity of any of the six soil zones. The soils are fine-grained with 87 to 98% of the material finer than the No. 200 sieve (<0.075 mm). Soils in zone 2 are similar, but contain greater percentages of sand and gravel; the fines content in zone 2 ranges between 60 and 93%, and averages 77.6%. Soils in zone 4 are similar to those of zone 2, but contain even greater percentages of sand with an average fines content of 69.9%.

Soil zones 3 and 5 represent the coarser-grained deposits at Site 3A; however, the fines content ranges considerably between samples. In soil zone 3, fines content ranges between 8 and 98%; the highest fines content was measured in a non-plastic silt at a shallow depth (14 to 16 ft bgs), and may represent a gradation of zone 2 soils. Average fines content in soil zone 3 was 43.1%, indicating a fairly high percentage of silt and clay. Average fines content in soil zone 5 is 20.3%, indicating a greater proportion of cleaner sand deposits.

Soils in zone 6 represent the Porters Creek Clay, where the fines content was found to range considerably between samples, from 15 to 96%. Average fines content was 40.6%, indicating that much of the Porters Creek Clay is actually a very fine sand with a high elastic silt content.

Summary of index properties testing. The results of the index property testing of soil samples from Site 3A are used to characterize the types of soils encountered in samples from each soil zone.

Soil zones 1 and 2 are similar in character. The soils are moderately wet, moderately plastic lean clays of medium density. The soils have a high fines content; soils in zone 1 average more than 94% fines and those in zone 2 average 78% fines, indicating that soil zone 2 contains slightly more sand than soil zone 1.

Soil zone 4, while similar to soil zone 2, contains some samples consisting of higher plasticity elastic silts with liquid limits as high as 106. The soils have a high fines content (70%), indicating slightly less proportion of sand as soils in zone 2. The soils are of medium density.

Soil zones 3 and 5 are similar in character to one another. The soils are relatively dry, clayey or silty sands. The fines content varies considerably between samples, with soils in zone 3 containing a higher portion of low-plasticity clay and soils in zone 5 containing a greater proportion of cleaner sand.

Soils in zone 6, the Porters Creek Clay, contain an average of 41% fines, indicating that much of the Porters Creek Clay is actually a very fine sand with a high elastic silt content. Moisture content of soils in zone 6 is high and plasticity is very high, with liquid limits up to 117. The specific gravity, void ratio, and density of the Porters Creek Clay are notably different from those of the overlying deposits and indicate that the soil is light weight and loose.

3.7.2.2 Fate and Transport Properties

Fate and transport properties are tests used to assess the contaminant release and migration behavior of a soil sample by describing its permeability and contaminant affinity to the soil mass. These results were subsequently used to support the development of the fate and transport model in Chap. 4.

Fate and transport properties tested for soils at Site 3A include permeability testing of relatively undisturbed Shelby tube samples and contaminant partitioning (or distribution) coefficients for selected radionuclides using split spoon samples. In addition, permeability was indirectly measured in the field using the results of pore pressure dissipation tests conducted at selected depth intervals in the SCPT soundings. Table 3.6 presents the average values of the geotechnical fate and transport properties as measured for each soil zone.

Table 3.6. Average fate and transport properties for each soil zone

Fate and transport property	Units	Soil zone 1	2	3	4	5	6
Hydraulic conductivity, *k*, (Shelby tube)	cm/sec	2.E-06	—	—	4.E-07	—	2.E-05
Hydraulic conductivity, *k*, (pore pressure dissipation)	cm/sec	—	—	3.E-05	—	5.E-05	—
		Across all soil zones					
Partitioning coefficient, K_d, (^{99}Tc)	L/kg	0.42 (0.17)					
Partitioning coefficient, K_d, (^{237}Np)	L/kg	133 (53)					

— indicates no samples tested from this soil zone.
Note: only arithmetic averages given in table.
K_d coefficients averaged across all soil zones; recommended conservative value is indicated in parentheses.

Permeability. Hydraulic conductivity is a measure of the relative rate that water flows through a soil under a unit hydraulic gradient, and is, therefore, indicative of the permeability of the soil. Similar to grain size distribution, the permeability is quite variable in soils at Site 3A, even within individual soil zones. Therefore, this discussion of permeability is a generalization of overall conditions and is not to be considered indicative of individual samples or lenses within a given soil zone. Vertical permeability values were measured in the laboratory and horizontal permeability values were estimated from results of field testing.

Vertical hydraulic conductivity was measured in the laboratory on selected Shelby tube samples from soil zones 1, 4, and 6. No Shelby tube samples were collected from soil zones 2, 3, or 5. No Shelby tube samples were collected from soil zone 2 because the materials could not be differentiated from soil zone 1 materials in the field; properties of soil zone 2 materials are expected to be similar to those of soil zone 1 based on their similar index properties. No Shelby tube samples were taken from soil zones 3 or 5 because those soil zones consist predominantly of sands and gravels from which collection of Shelby tube samples is not appropriate.

In soils from zone 1, the upper loess zone, hydraulic conductivity values range from 1.4E-07 to 9.3E-06 cm/sec, and average 2.E-06 cm/sec, which is typical of a low-permeability silty clay. In soils from zone 4, the lower silt and clay zone, hydraulic conductivity values are even lower, ranging from 3.2E-08 to 2.E-06 cm/sec, and averaging 4.E-07 cm/sec. As a result, soils in zone 4 would be expected to impede vertical movement of permeating groundwater. In soils from zone 6, the Porters Creek Clay, hydraulic conductivity values are highly variable, ranging from 5.4E-08 cm/sec to as high as 5.0E-05 cm/sec. The arithmetic average hydraulic conductivity in soil zone 6 is 2.E-05 cm/sec, and the geometric average is 3.E-06 cm/sec. The geometric average would be characteristic of the Porters Creek Clay for vertical movement of groundwater. It should be noted that because of the high stiffness of the Porters Creek Clay, which resulted in most of the Shelby tube samples reaching refusal penetration at more than 1000 psi of hydraulic pressure, the Shelby tube samples may be somewhat disturbed, which could result in measurement of higher hydraulic conductivities in the laboratory than representative of the undisturbed soil.

Horizontal hydraulic conductivity was estimated using results of field testing. Estimates of hydraulic conductivity for the coarser-grained deposits (silty sands and gravels) observed at Site 3A were calculated based on the results of the pore pressure dissipation tests in SCPT soundings (Appendix E). Test results were analyzed by plotting pore pressure versus square root of time, then identifying the time to achieve 90% equilibrium (t_{90}), in accordance with the Taylor Method (Winterkorn and Fang 1975). Time to achieve 50% equilibrium (t_{50}) was then calculated using rate of consolidation theory for Case 3 (sine curve initial pore pressure). Empirical relationships (Lunne, Robertson, and Powell 1997) were then used to estimate horizontal hydraulic conductivity based on the t_{50} time derived from the square root of time plot. Test results were also analyzed by identifying time to achieve 50% equilibrium (t_{50}) in accordance with the Cassagrande Method (Winterkorn and Fang 1975). Empirical relationships (Parez and Fauriel 1988) were used to estimate horizontal hydraulic conductivity based on the t_{50} time derived from the log of time plot. In addition, because the cone penetrometer used in this study had a tip area of 15 cm^2, all hydraulic conductivity values were multiplied by a correction factor of 1.5.

Table 3.7 summarizes these results. Horizontal hydraulic conductivities for these coarser-grained deposits are calculated to range from 2E-06 to 2E-04 cm/sec, which would be expected for a silty sand or silt, and are, therefore, somewhat lower than would be expected for clean sand or gravel. The average hydraulic conductivity for soil zone 3 is similar to that of soil zone 5 (3E-05 and 5E-05 cm/sec, respectively). Hydraulic conductivities are expected to vary widely across the site, depending on fines content in the deposit.

Table 3.7. Estimates of horizontal hydraulic conductivity from pore pressure dissipation tests

Test location	Depth (m)	Depth (ft)	t_{90} (sec)[a]	t_{50} (sec)	k (cm/s)[b]	t_{50} (sec)[c]	k (cm/s)[d]
SC01	5.5	18.0	149	45	4 E-06	52	1 E-05
SC05	5.36	17.6	128	39	5 E-06	33	2 E-05
SC06	4.93	16.2	90	27	7 E-06	42	1 E-05
	7.68	25.2	289	87	2 E-06	72	7 E-06
SC08	9.16	30.0	161	48	4 E-06	48	1 E-05
	15.56	51.0	25	8	3 E-05	5	2 E-04
SC09A	13.28	43.6	52	16	1 E-05	21	3 E-05
	14.64	48.0	104	31	6 E-06	30	2 E-05
SC10	10.68	35.0	21	6	3 E-05	12	7 E-05
	11.57	37.9	42	13	2 E-05	19	4 E-05
SB07	5.4	17.7	13	4	5 E-05	5.2	2 E-04
Arithmetic average					1.5 E-05		5.6 E-05
Geometric average					9 E-06		3 E-05

[a] t_{90} derived using Taylor square-root-of-time method.
[b] Source: (Lunne, Robertson, and Powell 1997).
[c] t_{50} derived using Cassagrande log-of-time method.
[d] Source: (Parez and Fauriel 1988).

Estimates of hydraulic conductivity for the finer-grained deposits (silts and clays) observed in SCPT soundings at Site 3A were not attempted because laboratory measurements were obtained. In many overconsolidated and fissured materials (such as those present at Site 3A), a dissipation test may first show an increase in pore pressure with time, reaching a peak value, and subsequent decrease in pore pressure with time. This type of response, termed dilatory dissipation, provides no clear 50% completion criterion, and the approach used for coarser-grained deposits is not applicable. Rigorous mathematical derivation based on cavity expansion-critical state solution to dilatory porewater decay with time has been developed, but relies on sample-specific dilation properties measured in the laboratory. Because finer-grained materials in Shelby tube samples were tested directly for permeability values in the laboratory, estimates based on dissipation tests are less reliable and, therefore, not attempted.

Partitioning Coefficient. The partitioning (or distribution) coefficient (K_d-value) is a measure of the relative concentration of a chemical or radionuclide in water to its concentration in the soil matrix, and is therefore indicative of the tendency of that constituent to either adsorb to the soil particles or to migrate through the soil. Partitioning coefficients were measured in the laboratory on selected split spoon samples from each of the six soil zones. These samples were tested using both ^{99}Tc and ^{237}Np solutions to determine their respective partitioning coefficients. These constituents were selected for testing because they have been found to be important contributors to risk as a result of contaminant migration in previous modeling at PGDP.

A total of 11 samples were selected for analysis. K_d values were determined based on distribution ratios derived using ASTM D4319 for 4 separate contact periods (3, 7, 10, and 14 days). Average distribution ratios for any given sample showed good correlation for 7 contact days and beyond. Therefore, the K_d-values reported here are the average distribution ratio values measured in the laboratory for contact periods of 7, 10, and 14 days.

The partitioning coefficients did not show any trend according to soil zone, nor did they show any trend according to percent fines content. Because inorganic contaminants like ^{99}Tc and ^{237}Np tend to adsorb to the fine-grained fractions within a soil sample, this lack of trend is unexpected.

Table 3.3 presents results for individual samples. Values for ^{237}Np showed wide variation, from a low of 14 L/kg to a high of 402 L/kg. The average value for ^{237}Np was 133 L/kg and the median was 79 L/kg. Because the data appear to be divided generally into two distinct ranges (one between 14 and 116 L/kg, and the other between 153 and 402 L/kg) the average and median values may be biased high. If only the lowest range of values were to be considered, then the average partitioning coefficient for ^{237}Np would be 53 L/kg. This is a conservative value that would be appropriate for use in fate and transport calculations for any contaminant migration modeling at Site 3A.

Values for ^{99}Tc also showed wide variation, from a low of 0.13 to a high of 1.42 L/kg. The average value for ^{99}Tc was 0.42 L/kg and the median was 0.20 L/kg. No K_d-value could be reported for 4 of the 11 samples tested, because there was no break-through in the permeant within the 14-day maximum contact period. The data again appear to be divided generally into two ranges: one between 0.13 and 0.20 L/kg, and the other between 0.32 and 1.42 L/kg. If only the lowest range of values were to be considered, then the average partitioning coefficient for ^{99}Tc would be 0.17 L/kg. This is a conservative value that would be appropriate for use in fate and transport calculations for any contaminant migration modeling at Site 3A.

Summary of fate and transport properties testing. The results of the fate and transport properties testing of soil samples from Site 3A are used to assess the contaminant release and migration behavior of soils encountered in samples from each soil zone.

Soil zones 1 and 2 are expected to be similar based on their similar index properties. Average vertical hydraulic conductivity (2.E-06 cm/sec) is typical of a low permeability silty clay.

Soil zone 4 contains high-plasticity elastic silts with vertical hydraulic conductivity values that are even lower (average 4.E-07) than soil zones 1 and 2. As a result, soils in zone 4 could be expected to impede vertical movement of groundwater.

Soil zones 3 and 5 show similar fate and transport characteristics to one another. Horizontal hydraulic conductivity values average 3.E-05 and 5.E-05 cm/sec, respectively, which would be expected for a silty sand or silt and, therefore, somewhat lower than would be expected for clean sand or gravel.

Soil zone 6, the Porters Creek Clay, demonstrates highly variable permeability, ranging from 5.E-08 to 5.E-05 cm/sec. The geometric average (3.E-06 cm/sec) would be characteristic for vertical movement of groundwater, typical of a low-permeability silty clay.

Partitioning coefficients showed wide variation between test samples and did not correlate with either the soil zone or fines content. A conservative estimate of the K_d-value for ^{237}Np is 53 L/kg and for ^{99}Tc is 0.17 L/kg. These conservative values are recommended for any subsequent fate and transport modeling for Site 3A.

3.7.2.3 Consolidation Properties

Consolidation properties are tests used to assess the amount and rate of consolidation of a soil sample under an applied load by describing its maximum historical soil load, load-settlement relationship, and time rate of consolidation. These results are subsequently used to support analysis of the settlement model in Chap. 4.

Consolidation, or compressibility, properties were measured using the results of one-dimensional consolidation tests on relatively undisturbed Shelby tube samples selected from soil zones 1, 4, and 6. No Shelby tube samples were collected from soil zones 2, 3, or 5 because soils in zone 2 could not be differentiated from soils in zone 1 in the field and because soils in zones 3 and 5 consist predominantly of sands and gravels. Table 3.8 presents the average values of the geotechnical consolidation properties as measured for each soil zone.

Table 3.8. Average consolidation properties for each soil zone

Consolidation property	Units	Soil zone 1	2	3	4	5	6
Hypothetical effective preconsolidation pressure, σ'_p	tsf	1.4	—	—	4.5	—	5.6
Existing effective pressure, σ'_v	tsf	0.5	—	—	1.7	—	2.7
Recompression index, C_r		0.03	—	—	0.03	—	0.09
Virgin compression index, C_c		0.12	—	—	0.15	—	0.36
Coefficient of consolidation, c_v	cm^2/sec	0.04	—	—	0.05	—	0.08

— indicates no samples tested from this soil zone.

Preconsolidation pressure. A soil is preconsolidated (or overconsolidated) if it has ever been subjected to a pressure in excess of its present overburden pressure. The previous pressure may have been caused by the weight of soil overburden that later eroded or by desiccation because of water level changes. The preconsolidation pressure is a measure of the maximum effective overburden pressure that likely existed in the past. The pressure is not measured directly, but is derived through graphical interpretation of the consolidation curve (plot of applied pressure versus void ratio), and is, therefore, an estimate of the hypothetical preconsolidation pressure.

For soils in zone 1, representative of the loess zone, the average preconsolidation pressure is 1.4 tsf. The average existing overburden pressure averages 0.5 tsf, approximately 0.9 tsf less than the preconsolidation pressure. For soils in zone 4, the average preconsolidation pressure is 4.5 and the average existing effective pressure is 1.7 tsf, approximately 2.8 tsf less than the preconsolidation pressure. For soils in zone 6, representative of the Porters Creek Clay, the average preconsolidation pressure is 5.6 tsf and the existing effective pressure averages 2.7 tsf, approximately 2.9 tsf less than the preconsolidation pressure. Based on these results, it can be inferred that the deeper soil zone 4 and 6 deposits exhibit a similar history of preconsolidation, but that much less preconsolidation is exhibited by the soil zone 1 loess deposits. An erosional sequence likely occurred at some point above the zone 4 soils, but below the zone 1 soils. This is consistent with the geologic description of Site 3A.

Compression index. The compression index is the slope of the consolidation curve and is a measure of the amount of consolidation that occurs under increasing loads. The virgin compression index is the slope of the consolidation curve at pressures higher than the preconsolidation pressure. The recompression index can be roughly inferred from the slope of the curve at pressures less than the preconsolidation pressure, or during unloading (rebound) of the testing apparatus. Because in the laboratory there is little difference in slope between rebound and recompression curves, it is assumed that field consolidation is roughly equal to the laboratory rebound curve. Because the recompression index is much smaller than the virgin compression index, errors are minimal. In settlement computations, precompressed layers can often be considered incompressible (Terzaghi and Peck 1948).

Soils in both zones 1 and zone 4 exhibit similar compression characteristics, despite their differences in preconsolidation. The average virgin compression index for soil zone 1 is 0.12 and for soil zone 4 is 0.15. The average recompression index for soil zone 1 is 0.03 and for soil zone 4 is 0.03. Soils in zone 6, representative of the Porters Creek Clay, exhibit much greater compressibility, albeit a much higher degree of preconsolidation. The average virgin compression index for soil zone 6 is 0.36, and the average recompression index is 0.09.

Coefficient of consolidation. Consolidation of a soil layer occurs gradually over time as the water drains out of the soil to relieve water pressures imposed by a constant load. The mechanics of the time delay are influenced by the permeability of the soil, the load applied, and the length and geometry of the water flow path through the soil. These mechanics are standardized in a consolidation test by using a standard geometry of the testing apparatus and by considering a standard degree of consolidation at a given time. The coefficient of consolidation is a measure of the time to achieve consolidation under a given load increment and is derived using solutions of differential equations for the testing geometry. It is, therefore, an indirect measure of the time it takes for consolidation to occur and inversely proportional to the degree of consolidation; that is, the higher the value the shorter the time to achieve the same degree of consolidation. The coefficient of consolidation was determined for each testing load increment using the Taylor square-root-of-time graphical method (Winterkorn and Fang 1975). The average coefficient of consolidation for a given sample was then determined by averaging the values within a range of load increments corresponding to the approximate existing overburden pressure.

For soil zone 1, the average coefficient of consolidation is 0.04 cm^2/sec, for soil zone 4 it is 0.05 cm^2/sec, and for soil zone 6 it is 0.08 cm^2/sec. The values vary considerably between samples and between test load increments, from less than 0.01 cm^2/sec to more than 0.15 cm^2/sec.

Summary of consolidation properties testing. The results of the consolidation properties testing of soil samples from Site 3A are used to assess the amount and rate of consolidation of soils encountered in samples form each soil zone.

Soil zone 1 demonstrates some preconsolidation, with a preconsolidation pressure averaging 1.4 tsf, and moderate compressibility, with a compression index of 0.12. The average coefficient of consolidation for soil zone 1 is 0.04 cm^2/sec, indicating a relatively fast rate of consolidation would be expected for this soil zone. Soil zone 2 would be expected to be similar to soil zone 1 based on their similar index properties.

Soil zones 4 and 6 exhibit a similar history of preconsolidation that is much higher than soil zone 1, inferring than an erosional sequence likely occurred at some point above soil zone 4 but below soil zone 1. The average coefficient of consolidation for these soil zones is somewhat higher, indicating a slightly slower rate of consolidation than soil zone 1. The compressibility of soil zone 4 is similar to that of soil zone 1, but soil zone 6, the Porters Creek Clay, is more than twice as compressible, with an average compression index of 0.36.

Soil zones 3 and 5 were not tested for consolidation properties because they are predominantly sands and gravels.

3.7.2.4 Strength Properties

Strength properties are tests used to assess the shear strength of a soil by describing its frictional and cohesional behavior under an applied load. These results are subsequently used to support analysis of the bearing capacity model in Chap. 4.

Soil strength properties were measured using the results of unconsolidated-undrained triaxial compression tests on relatively undisturbed Shelby tube samples selected from soil zones 1 and 4. No Shelby tube samples were collected from soil zones 2, 3, or 5 because soils in zone 2 could not be differentiated from soils in zone 1 in the field and because soils in zones 3 and 5 consist predominantly of sands and gravels. Shelby tube samples collected from soil zone 6 were either of insufficient volume or exhibited disturbance (fracturing) during sampling because of the high stiffness of the Porters Creek Clay and could not be tested. Table 3.9 presents the average values of the geotechnical strength properties as measured for each soil zone.

Table 3.9. Average strength properties for each soil zone

Strength property	Units	Soil zone					
		1	2	3	4	5	6
Apparent friction angle, φ	degrees	9.5	—	—	6.3	—	N/A
Apparent cohesion, c, (U-U tests)	tsf	0.95	—	—	1.5	—	N/A

— indicates no samples tested from this soil zone.
N/A = not available.

Unconsolidated-Undrained Shear Strength. Unconsolidated-undrained shear strength parameters measure the shear strength of the soil in a condition under which the stresses are changed so rapidly with respect to the ability of the soil to drain that no dissipation of pore pressure takes place. These extreme conditions are rarely realized in the field, but they represent conservative limiting conditions of soil strength. Unconsolidated-undrained shear strength is used in a total stress analysis, disregarding the effective stress reduction because of pore pressure increases during loading.

The shear strength of a soil under varying normal stresses is represented on a Mohr's rupture diagram (plot of normal stress versus shear stress). The rupture line is derived graphically by interpreting an envelope surrounding multiple failure circles. For this investigation, the laboratory typically performed the U-U test using three separate confining loads, resulting in three failure circles for each sample. The shear strength envelope is represented by two parameters: the apparent friction angle and the apparent cohesion. The apparent friction angle, φ, is the slope of the rupture line and the apparent cohesion, c, is the intercept of the rupture line (shear stress at the point where normal stress is zero). For clays, the friction angle is generally zero, so that the shear strength is constant under all loads. For clean sands and gravels, the cohesion is generally zero, so that the shear strength increases under increasing load. Soils between these two extremes exhibit characteristics of both.

Soil zone 1 represents the upper loess deposits and consists of lean clays of moderate plasticity. The strength parameters (φ and c) varied considerably between individual tests. Apparent friction angle, φ, varied between 1.7 and 16.3°, averaging 9.5°. The apparent cohesion, c, varied between 0.0 and 2.1 tsf, averaging 0.95 tsf. Soil zone 4 represents deeper elastic silts and lean clays. The apparent friction angle varied between 3.3 and 10.8°, averaging 6.3°. The apparent cohesion varied between 0.7 and 2.6 tsf, averaging 1.5 tsf.

Summary of strength properties testing. The results of the strength properties testing of soil samples from Site 3A are used to assess the shear strength of soils encountered in samples from each soil zone. For the Seismic Assessment, strength properties were measured under unconsolidated-undrained conditions that would be expected under relatively quick loading.

Soil zones 1 and 4 demonstrated similar strength characteristics, with low apparent friction angles (average 9.5° and 6.3°, respectively) and moderately high apparent cohesion (average 0.95 and 1.5 tsf, respectively). Soil zone 2 would be expected to be similar to soil zone 1 based on its similar index properties and similar results of field testing (N'_{60}-values).

Soil zone 6 samples could not be tested because there was either insufficient volume or disturbance (fracturing) in the sample. Estimated strength parameters for soil zone 6 are discussed further in Chap. 4 in development of the bearing capacity model.

Soil zones 3 and 5 were not tested for strength properties because they are predominantly sands and gravels from which Shelby tube samples could not be taken.

3.8 SEISMIC SOIL PROPERTIES AT SITE 3A

Seismic properties were tested in the field at Site 3A to assess the shear-wave velocity of the soil profile. These results are subsequently used in the seismic design model in Chap. 7 to derive PGA and design ground motions. Seismic velocities (both compression and shear) were measured directly in the field in the seismic velocity log of deep borehole DB-02 and in the SCPT soundings.

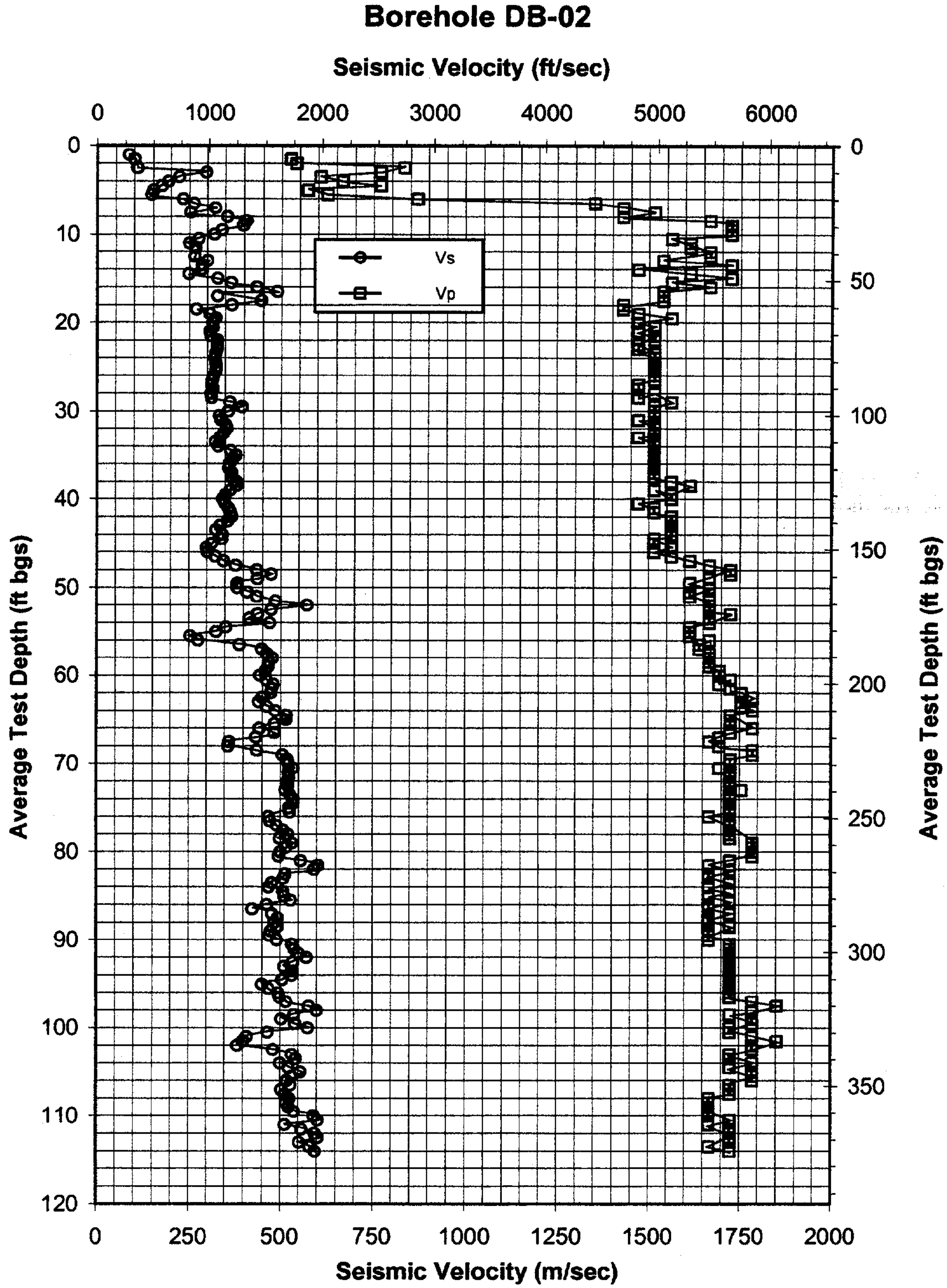
Seismic compression-wave velocities measured at DB-02 are presented in Appendix E and in Fig. 3.24. As shown in Fig. 3.24, compression-wave velocities generally increase with depth, although lower compression-wave velocities were measured in the Porters Creek Clay. Compression-wave velocities in the uppermost loess deposits were between 1500 and 3000 fps. Compression wave velocities within the Terrace Deposits (to a depth of 60 ft bgs in DB-02) were between 4500 and 5500 fps. Compression-wave velocities within the Porters Creek Clay (to a depth of 155 ft bgs in DB-02) were between 4900 and 5300 fps and within the McNairy Formation were between 5300 and 6000 fps.

Seismic shear-wave velocities measured at DB-02 and in the SCPT soundings are presented in Appendix E. Figure 3.24 presents the shear-wave velocities at DB-02. As with the compression-wave velocities, the shear-wave velocities shown in Fig. 3.24 generally increase with depth. Figure 3.25 summarizes the shear-wave velocities obtained from all soundings and from depths up to 70 ft bgs. The data show a generally increasing shear-wave velocity with depth. However, the data also show fairly wide scatter across all SCPT soundings.

Figure 3.26 summarizes the shear-wave velocities obtained from all soundings up to 70 ft bgs grouped by soil zone. The plots shown on Fig. 3.26 show generally increasing shear-wave velocity with depth, and marked differences in average shear-wave velocity between soil zones. Table 3.10 summarizes the average shear-wave velocity value and standard deviation by soil zone. For soil zone 1, which consists of shallow silty and clayey loess deposits, shear-wave velocities are relatively low, ranging between 400 and 800 fps. For soil zones 2 and 4, which consist predominantly of alluvial silts and clays, shear-wave velocity values generally range from 600 to 1200 fps. For soil zone 6, representing the deeper silts and clays of the Porters Creek Clay, shear-wave velocities generally range from 800 to 1300 fps. Data are broadly scattered for soil zones 3 and 5, which represent loose to very dense sand and gravel deposits. Shear-wave velocities for these coarser-grained materials range from as low as 800 fps to as high as 1900 fps.

Table 3.10. Summary of seismic shear-wave velocity results for each soil zone

Soil zone	Average seismic shear-wave velocity, Vs (ft/sec)	Standard deviation of seismic shear-wave velocity, Vs (ft/sec)
1	607	162
2	921	208
3	1250	352
4	913	159
5	1068	251
6	1028	202



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Vs ....Shear (s–wave) velocity
Vp ....Compression (p–wave) velocity

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Fig. 3.23. Seismic velocity log for borehole DB-02.

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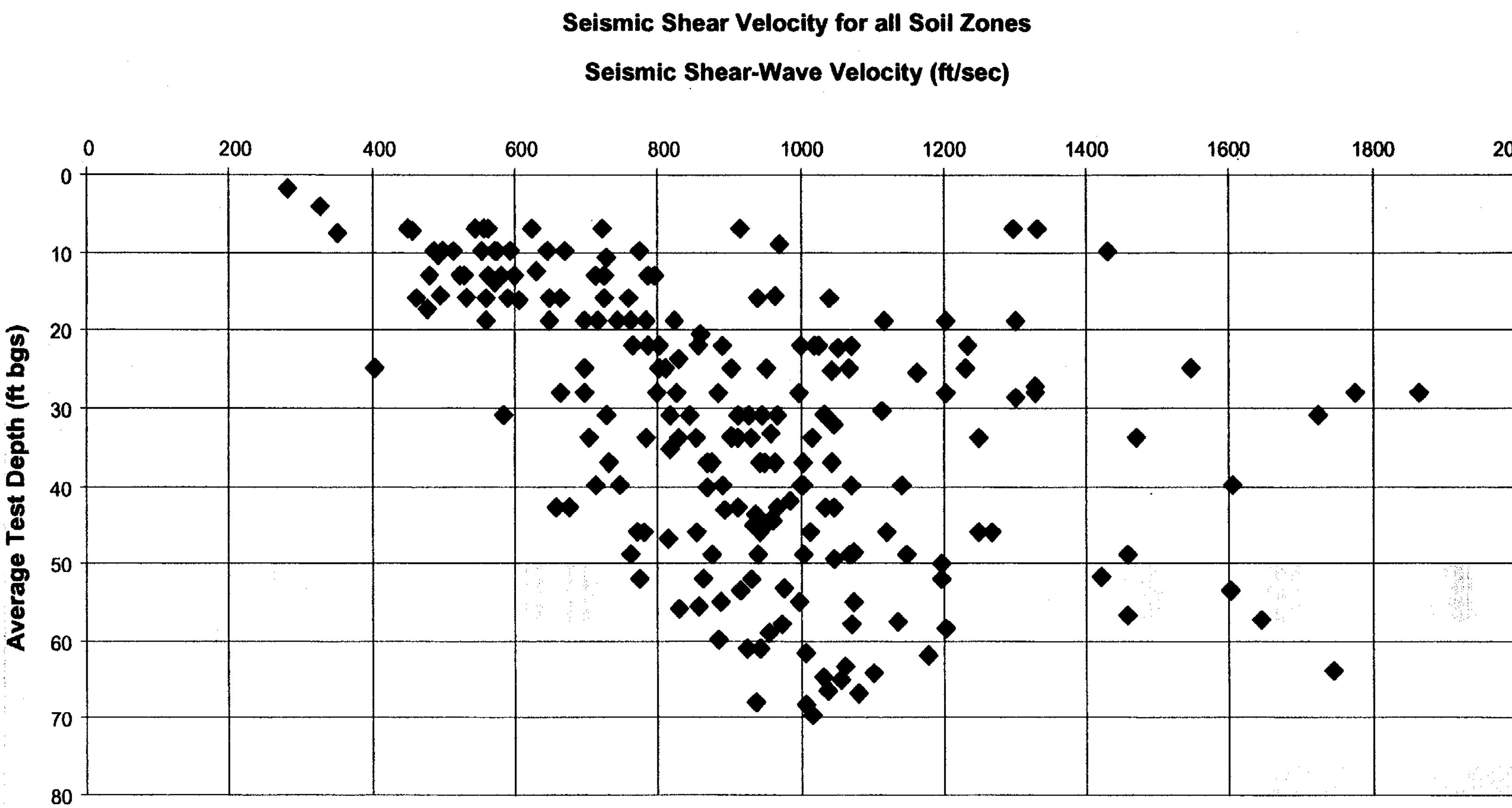

Fig. 3.24. Plot of seismic shear-wave velocity vs. depth for all soil zones.

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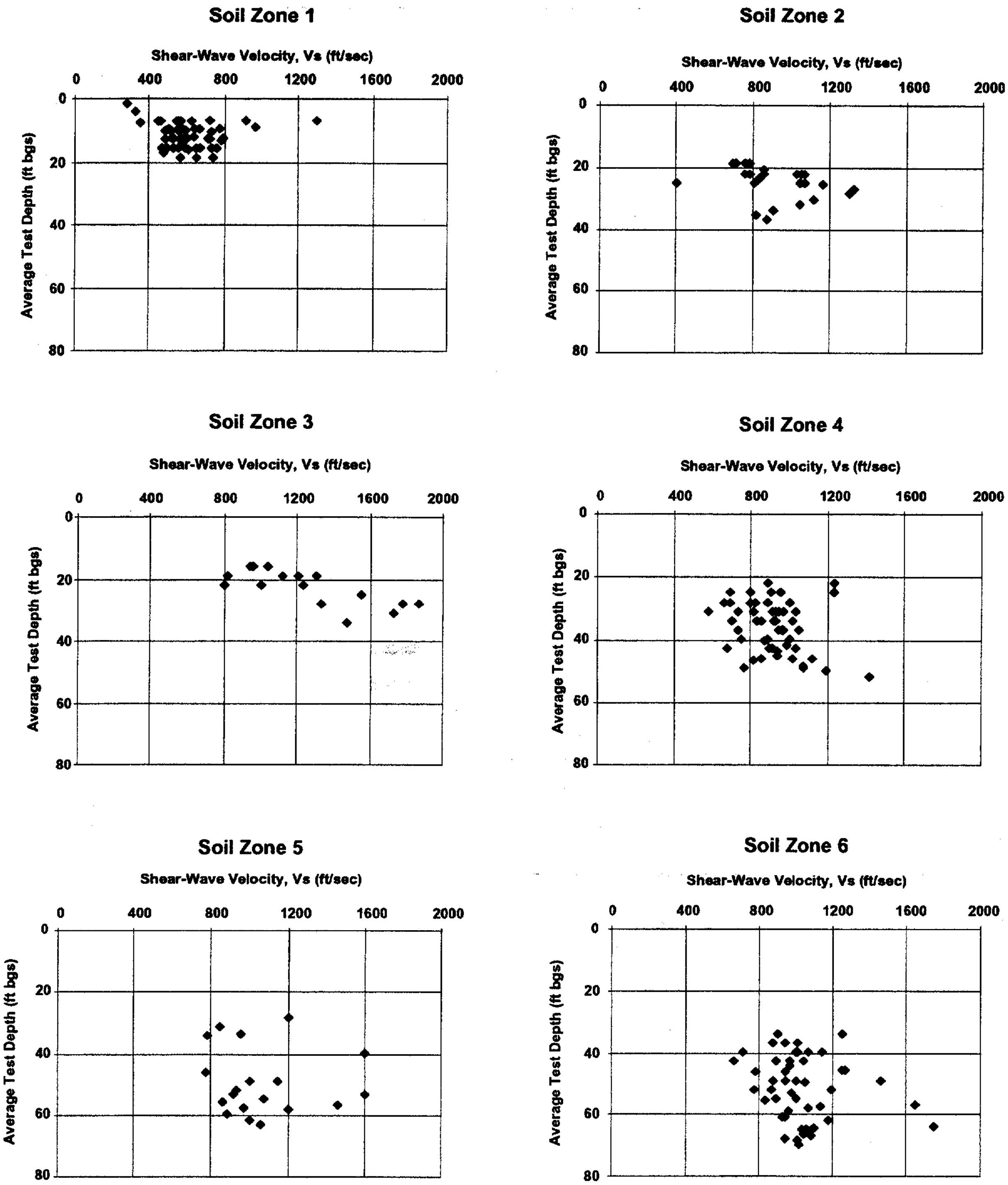

Fig. 3.25. Plots of seismic shear-wave velocity vs. depth for different soil zones.

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3.9 SUMMARY

This chapter described the physical characteristics of the subsurface at PGDP based upon results of the field characterization completed at Site 3A.

Bedrock was encountered at a depth of 400 ft bgs in borehole DB-02 at Site 3A. The bedrock surface varies between 325 to as much as 425 ft bgs. The McNairy Formation was encountered overlying the bedrock, below a depth of approximately 155 ft bgs between 30 and 60 ft bgs. The Porters Creek Clay was encountered, in turn, overlying the McNairy Formation. Depth to the top of the Porters Creek Clay varied. Terrace Deposits typically overly the Porters Creek Clay to a depth of 15 to 20 ft bgs. Surficial loess deposits were encountered overlying the Terrace Deposits. In some areas, younger alluvial deposits of Holocene age fill former erosional features incised in the loess.

Soils within the uppermost 70 ft of the soil profile were grouped into six different soil zones for purposes of interpreting geotechnical and seismic soil properties at Site 3A. These properties, including index, fate and transport, consolidation, strength, and seismic properties, are used in subsequent chapters of this report in developing a geotechnical design model, evaluating liquefaction potential, and developing a seismic design model for Site 3A.

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4. GEOTECHNICAL DESIGN MODEL

Chapter 3 described the physical characteristics of Site 3A and presented the results of the field and laboratory testing conducted during the Seismic Investigation. This chapter develops a geotechnical design model of the site conditions, using the information and designated soil zones presented in Chap. 3. Average geotechnical properties for each soil zone are summarized in Table 4.1. Geotechnical design considerations regarding fate and transport, settlement, and bearing capacity of a potential on-site CERCLA waste disposal facility are discussed. This information would be used in any subsequent evaluation of the technical feasibility and effectiveness of such a facility.

Table 4.1. Average geotechnical properties for each soil zone

Geotechnical property	Units	Soil zone 1	2	3	4	5	6
Index properties							
Moisture Content, w	%	24	20	15	20-35	19	56-59
Liquid Limit, LL	%	32	32	25	56	28	89
Plastic Limit, PL	%	19	15	14	28	14	60
Specific Gravity, $S.G.$	—	2.66	2.60	2.50	2.57	2.67	2.43
Dry Density, γ_d	pcf	100.8	—	—	109.6	—	59.8
Initial Void Ratio, e_0	—	0.69	—	—	0.55	—	1.89
Fines Content, $P200$, (< 0.075 mm)	%	94.4	77.8	43.3	74.2	20.3	43.0
Consolidation properties							
Hypothetical effective preconsolidation pressure, σ'_p	tsf	1.4	—	—	4.5	—	5.6
Existing effective pressure, σ'_v	tsf	0.5	—	—	1.7		2.7
Recompression index, C_r	—	0.03	—	—	0.03	—	0.09
Virgin compression index, C_c	—	0.12	—	—	0.15	—	0.36
Coefficient of consolidation, c_v	cm^2/sec	0.04	—	—	0.05	—	0.08
Shear strength properties							
Apparent friction angle, φ	degrees	9.5	—	—	6.3	—	N/A
Apparent cohesion, c, (U-U tests)	tsf	0.95	—	—	1.5	—	N/A
N'_{60}-value (from SPTs and SCPTs)	blows/ft	17-18	12-20	29-46	13-15	23-26	25-32
Fate and transport properties							
Hydraulic conductivity, k, (Shelby tube)	cm/sec	2.E-06	—	—	4.E-07	—	2.E-05
Hydraulic conductivity, k, (pore pressure dissipation)	cm/sec	—	—	3.E-05	—	5.E-05	—
		Across all soil zones					
Partitioning coefficient, K_d (^{99}Tc)	L/kg	0.42 (0.17)					
Partitioning coefficient, K_d (^{237}Np)	L/kg	133 (53)					

K_d coefficient averaged across all soil zones; recommended conservative value is indicated in parentheses.
N'_{60} = corrected SPT blow counts
N/A = not available
SCPT = Seismic Cone Penetrometer Test
SPT = Standard Penetration Test
U-U = unconsolidated-undrained

4.1 BACKGROUND – CONCEPTUAL FACILITY DESIGN

A potential on-site CERCLA waste disposal facility could be designed to accommodate varying waste disposal strategies. Under a comprehensive strategy for CERCLA waste disposal, a facility could be designed to handle waste volumes generated through ultimate decontamination and decommissioning of PGDP, including nonhazardous and nonradioactive solid waste, as well as hazardous and/or radioactive waste meeting the waste acceptance criteria established for the facility. This comprehensive strategy would manage approximately 3,100,000 yd^3, which is estimated to be the greatest volume of waste that could be considered for placement in a potential on-site CERCLA waste disposal facility (DOE 2001). Other strategies would manage lesser volumes of waste. Therefore, this geotechnical design model has been developed considering a facility designed to accommodate 3,100,000 yd^3. Allowing approximately 10% of the volume for interim soil cover, such a facility would need to have an overall capacity of 3,400,000 yd^3.

The facility would be constructed as an above-ground disposal cell with ancillary facilities. Based on the projected comprehensive waste volumes and conceptual facility design, the waste disposal facility would require a total area of at least 110 acres. Of that, the disposal cell would occupy approximately 32.5 acres, and the containment dike would occupy another 51.5 acres. Support facilities, perimeter roads, ditches, and buffer zones would occupy the remaining 26 acres.

The conceptual design of the disposal cell is shown on Fig. 4.1. The disposal cell would likely include the following components:

- a clean-fill perimeter dike, which would be an embankment approximately 40 ft abovegrade, 100 ft wide at the crest, with 6:1 (horizontal:vertical) side slopes;
- a geologic buffer of native recompacted loess;
- a 6-ft-thick multilayer base liner system consisting of primary and secondary flexible membrane and clay liners, primary and secondary leachate collection/detection systems, and a protective soil layer; and
- a 16-ft-thick multilayer cap consisting of a low-permeability clay liner, a flexible membrane liner, a drainage layer, a biointrusion layer, and a soil-rock matrix cover.

The waste fill would be placed within the clean-fill dike to a total thickness of approximately 80 ft. The total height of the disposal cell, including base liner and cap, would therefore be approximately 102 ft above existing grade (6 ft base liner, 80 ft waste, 16 ft cover).

The waste forms that may be considered for disposal in the potential on-site CERCLA waste disposal facility could include soil, sediment, concrete, scrap metal, building demolition debris, and other dry waste forms. The bulk unit weight of such materials is estimated to average approximately 115 pcf.

4.2 FATE AND TRANSPORT MODEL

The conceptual site model for fate and transport is a statement of known site conditions that serves as the framework for fate and transport modeling. These site conditions include hydrogeologic and transport properties. Site-specific geology, hydrogeology, and geotechnical soil properties are presented in Chap. 3 and summarized below. Previous groundwater flow model development for other areas at PGDP has added insight into aquifer properties and transport parameters.

4.2.1 Hydrogeologic Properties

Soil zone 6, the Porters Creek Clay, is generally encountered at depths below 30 to 60 ft bgs. The surface elevation of the top of the formation at Site 3A ranges from 365 to 330 ft msl. The Porters Creek Clay is a firm to hard laminated silt and clay of low to very high plasticity. Because the formation has a low permeability (arithmetic average 2.E-05 cm/sec and geometric average 3.E-06 cm/sec) and is approximately 90 ft thick at Site 3A, it represents a barrier to vertical (downward) migration of groundwater. The primary hydrogeologic units of concern at Site 3A, therefore, occur within the Terrace Deposits overlying the Porters Creek Clay.

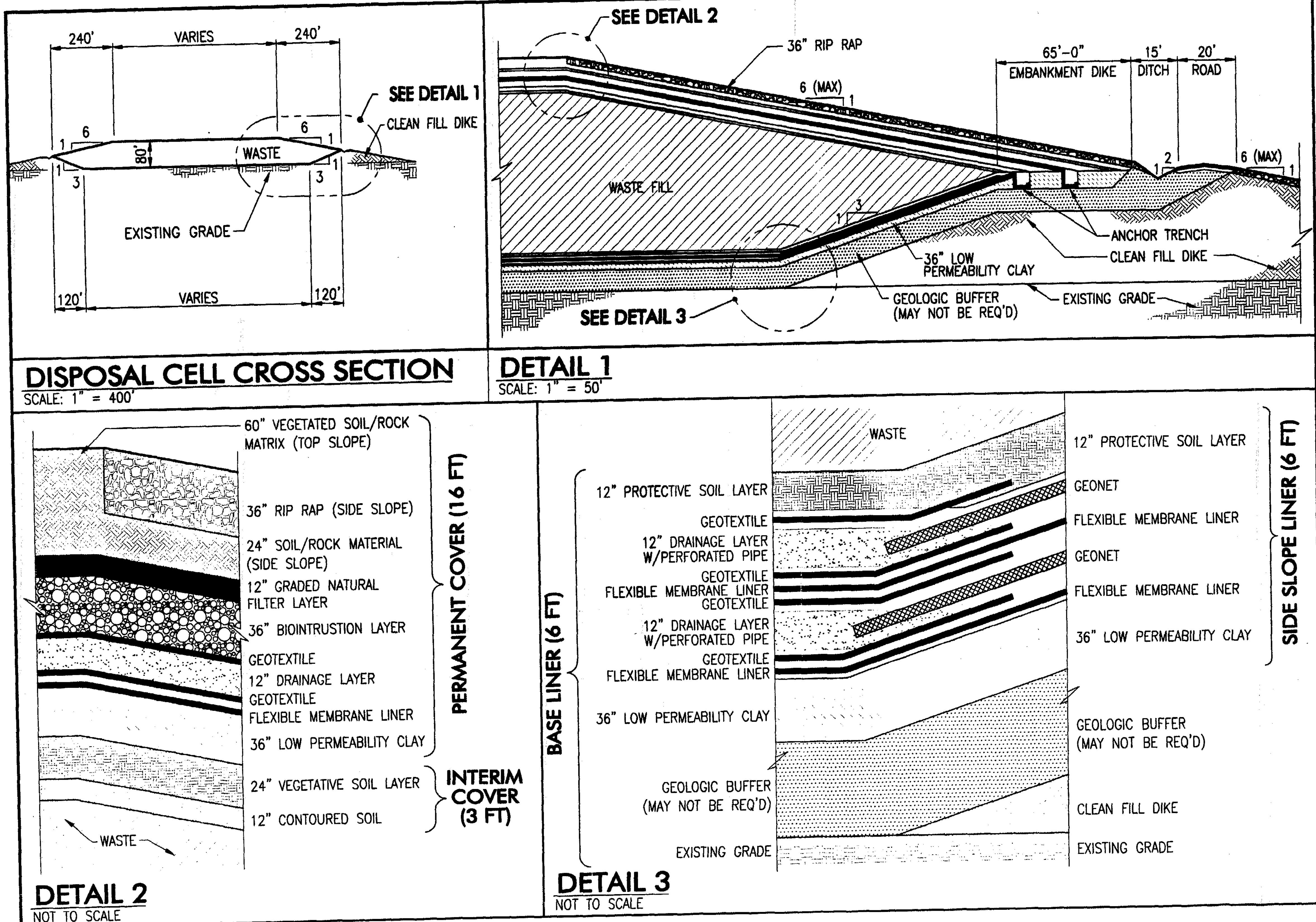

Fig. 4.1. Conceptual cross section of disposal cell.

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Terrace Deposits are typically present below a depth of 15 to 20 ft bgs. The thickness of the deposits varies between 20 to 50 ft. The deposits vary considerably across the site in texture and stiffness and are present in beds that are not laterally continuous across the site. As such, the unit is designated as four separate geotechnical soil zones for interpreting soil properties. The following generally describe the zones.

Soil zone 5 consists of a lower sand unit generally present below a depth of 32 ft bgs and directly overlying the Porters Creek Clay. The unit consists generally of a fine to medium, poorly graded sand grading downward to a well-graded sand and gravel. Index property testing indicates low fines content (average 20%) and variable USCS classifications of SP-SM and SC. Results of pore pressure dissipation tests suggest an average hydraulic conductivity of 5.E-05 cm/sec for soil zone 5.

Soil zone 4 consists of interbedded silts and clays with occasional poorly graded sand lenses. The deposits typically overly zone 5 deposits, but directly overly the Porters Creek Clay at SB-05. The soils are moderately to highly plastic silts and lean clays (USCS classification MH or CL) that are moderately dense, with low permeability averaging 4.E-07 cm/sec.

Soil zone 3 consists of an upper sand unit generally present above a depth of 32 ft bgs and often directly underlying the surficial loess deposits. The sands have relatively high fines content (average of 43%) and variable USCS classifications of SC, ML, CL, and GW-GC. Results of pore pressure dissipation tests suggest an average hydraulic conductivity of 3.E-05 cm/sec for soil zone 3.

Soil zone 2 consists of an upper silt and clay unit directly underlying the surficial loess deposits. The unit is similar to both soil zones 1 and 4, with an average of 78% fines and moderate plasticity.

Soil zone 1 (loess) consists of low to moderately plastic silts or lean clays (USCS classification CL). The soil is moderately dense, with a low permeability averaging 2.E-06 cm/sec.

The Terrace Deposits Flow System is a water table flow system that has developed in these units. The Terrace Deposits Flow System provides some throughflow to the north, across the Porters Creek Clay terrace, ultimately recharging the Upper Continental Deposits beneath the plant. Results of pore pressure dissipation tests and previous investigations suggest the water table within the Terrace Deposits Flow System is present at depths of 5 to 17 ft bgs. Flow is generally to the northwest, at a horizontal gradient of approximately 0.009 ft/ft, discharging to the UCRS beneath the plant. Because of their higher permeabilities, flow within the water table is predominantly within soil zones 3 and 5.

The water table fluctuates throughout the year in response to rainfall. At times of the year when the water table is higher, flow within the Terrace Deposits Flow System may discharge locally to tributaries of Bayou and Little Bayou Creeks.

A water balance accounts for the components of the hydrologic cycle of a site and estimates the amount of rainfall that returns to the atmosphere as evapotranspiration, exits the site as surface runoff, or infiltrates the site as groundwater recharge. Groundwater recharge is the amount available to promote contaminant migration away from a waste source. Previous groundwater flow models at PGDP have provided an estimate of the groundwater recharge budget. The annual rainfall for the PGDP averages 50 inches/year. Of this rainfall total, approximately 4.6 inches of water infiltrate through the UCRS to the RGA.

4.2.2 Transport Properties

Partitioning coefficients (K_d-values) were determined for this Seismic Investigation for ^{237}Np and ^{99}Tc. These data showed wide variation between test samples and did not correlate with either the soil zone or fines content. Conservative estimates of the respective K_d-values are recommended for use in fate and transport calculations for any contaminant migration modeling at Site 3A. These estimates were developed considering only the lowest range of values measured in the laboratory for each constituent. A value of 57 L/kg is recommended for ^{237}Np and a value of 0.17 L/kg is recommended for ^{99}Tc.

4.2.3 Assumed Contaminant Release Mechanisms and Migration Pathways

A portion of rainwater falling on the surface of a waste disposal cell would infiltrate through the cover materials into the disposed wastes and is thereby expected to produce leachate. The primary mechanism of contaminant migration is expected to be percolation of rain water, which would subsequently react with the disposed wastes and produce leachate (i.e., dissolved-phase contaminants in equilibrium with source materials). Four processes causing the reactions may be considered: solubility limited, surface rinse, diffusion, and uniform release (dissolution). Thereafter, the leachate would migrate downward into the leachate collection system, and would be removed from the disposal cell. For the purpose of this discussion, it is assumed that the leachate collection system would no longer function and that the leachate, rather than being collected and removed, would migrate out of the bottom of the cell, through the unsaturated zone, and into the water table. Once a contaminant reaches the water table it would be expected to migrate with groundwater through advection and dispersion to the receptor location. The Terrace Deposits Flow System discharges to the UCRS beneath the plant, and the UCRS in turn recharges the RGA. Groundwater in the RGA ultimately migrates beyond the DOE property boundary discharging to the Ohio River.

4.3 SETTLEMENT MODEL

Because the disposal cell would be constructed above ground, settlement of the base liner system can be expected to occur as a result of consolidation of the underlying clayey and silty soil zones. As the disposal cell is filled, the weight of the waste, liner, and cover materials would cause an increase in effective pressure at considerable depth. The soil would respond to such increased pressure by consolidating, or compressing, in accordance with its specific consolidation properties, as presented in Sect. 3.7.2.3. This response is depicted in a consolidation curve (plot of applied pressure vs. void ratio) as measured in the geotechnical laboratory; consolidation curves for samples collected in this Seismic Investigation are presented in Appendix E and summarized in Sect. 3.7.2.3.

For the potential disposal cell, the total height of fill would be 102 ft at a unit weight of 115 pcf, which would result in a pressure increase of 5.87 tsf. Because the disposal cell would cover more than 84 acres, it is assumed that this load would be felt throughout the depth of the soil profile to the bottom of the Porters Creek Clay. Because the McNairy Formation consists predominantly of sands, settlement of the McNairy Formation is disregarded in this settlement model. The following additional assumptions have been made in developing the settlement model.

- The water table is assumed to be present at a depth of 15 ft bgs.

- The loess (soil zone 1) is assumed to be left in place and is not removed or recompacted. This results in a conservative estimate of settlement. Soil zone 1 is assumed to be 20 ft thick. The existing effective pressure at the midpoint of soil zone 1 is 0.50 tsf, based on the average index properties presented in Table 4.1.

- Elastic silt and lean clay (soil zone 4) is assumed to underlie soil zone 1 and to be 30 ft thick. Soil zones 2, 3, and 5 are disregarded for this analysis. Soil zone 2 may be disregarded because it is similar in its properties to both soil zones 1 and 4. Soil zones 3 and 5, which contain greater proportions of sands and gravels, are likely to be less compressible than soil zone 4. Therefore, soil zones 3 and 5 may be disregarded as a conservative estimate of settlement. The existing effective pressure at the midpoint of zone 4 is 1.5 tsf.

- The Porters Creek Clay (soil zone 6) is assumed to underlie soil zone 4 and to be 100 ft thick. The Porters Creek Clay was modeled as two separate layers, each 50 ft thick, so that settlement could be estimated more accurately. The existing effective pressures at the midpoints of these two layers are 2.5 and 3.3 tsf, respectively.

- All layers are assumed to exhibit behavior consistent with their average consolidation properties, as presented in Table 4.1.

- The McNairy Formation is assumed to be incompressible.

Settlement was calculated for each soil zone assuming recompression occurs for loads up to its respective preconsolidation pressure, and that virgin consolidation occurs for the remaining load. The average preconsolidation pressure for each soil zone is presented in Table 4.1. Settlement was calculated using the following general equation (Peck, Hansen, and Thornburn 1974):

$$S = C_c / (1+e_0) * H * \log_{10}((\sigma'_v + \Delta p)/ \sigma'_v),$$

where

S = estimated settlement in ft,
C_c = consolidation index,
e_0 = void ratio,
H = thickness of soil layer in ft,
σ'_v = existing effective overburden pressure in tsf, and
Δp = change in pressure in tsf.

Table 4.2 presents the results of the settlement calculations for the 102-ft high disposal cell. Total settlement is estimated to be as much as 5.2 ft in the center of the disposal cell area. Nearly 3.2 ft of the total settlement is estimated for the Porters Creek Clay. As discussed in Sect. 3.6.2, the high stiffness of the Porters Creek Clay resulted in most of the Shelby tube samples reaching refusal penetration, which could have disturbed the samples. Sample disturbance could result in higher consolidation ratios and void ratios and lower unit weights for the Porters Creek Clay samples. This could result in an overestimation of the actual predicted settlement.

Table 4.2. Estimated consolidation settlement of 102-ft high fill at Site 3A

Soil zone	Thickness (ft)	Reconsolidation settlement (ft)	Virgin consolidation settlement (ft)	Total consolidation settlement (ft)
1	20	0.16	0.93	1.09
4	30	0.27	0.68	0.95
6a	50	0.52	1.19	1.71
6b	50	0.43	1.06	1.49
TOTAL	**150**	**1.38**	**3.86**	**5.24**

Differential settlement across the disposal cell would likely occur due to variations in soil properties, thickness of underlying soil zones, and distance from the perimeter of the clean-fill dike. Differential settlements have not been modeled in this analysis because of the numerous uncertainties involved, but are anticipated to be as large as 2 to 3 ft across the disposal cell. Detailed design of the disposal cell would need to account for such differential settlement by increasing the slopes of the base grades, bottom liner, and drain lines, and by selecting appropriate construction materials.

As discussed in Chap. 3, consolidation of a soil layer occurs gradually over time as the water drains out of the soil to relieve water pressure imposed by the increased load. The time needed to achieve 90% of the total consolidation predicted above was modeled using the average consolidation properties presented in Table 4.1 and the following assumptions:

- Two-way vertical drainage was assumed. The total drainage length was assumed to be half the depth to the bottom of the Porters Creek Clay, or 75 ft. The effect of drainage that may occur within the sand and gravel layers (soil zones 3 and 5) were disregarded. This is reasonable because those layers are not uniform across the site, they are relatively thin layers, and the distance to the surrounding drainage discharge points is large (greater than 1000 ft).

- To simplify the analysis, the coefficient of consolidation for only the Porters Creek Clay was used. This may slightly underestimate the time of consolidation.

The time to achieve 90% consolidation was calculated using the following general equation (Peck, Hansen, and Thornburn 1974).

$$t_{90} = T_{90} * H^2 / c_v,$$

where

t_{90} = estimated time to achieve 90% consolidation in years,
T_{90} = theoretical time factor for two-way drainage and 90% consolidation = 0.848,
H = total drainage length = 75 ft, and
c_v = coefficient of consolidation in ft^2/yr.

The calculated t_{90} is 1.8 years. Because the disposal cell would be filled gradually over a 20-year period of operation, most of the predicted consolidation settlement would be essentially completed by the time the cell is filled. Therefore, if any undesirable effects on base grades or drain lines were to be caused by the total or differential settlement, these effects could be observed during the period of operation of the waste disposal facility and corrective measures would be implemented prior to closure.

4.4 BEARING CAPACITY MODEL

When a load is applied on a portion of the ground surface, the surface settles. Once that load reaches a critical load, the soil is no longer capable of supporting the load without failure, and the foundation breaks or punches into the ground. This critical load is referred to as the bearing capacity of the foundation. It is dependant on the size and shape of the foundation; the composition of the supporting soil; and the character, rate, and frequency of the loading. Every foundation should be capable of supporting, with a reasonable margin of safety, the maximum load to which it is ever likely to be subjected. For this reason, foundations are designed so as to possess a certain factor of safety against bearing capacity failure of the soil.

Foundation bearing capacity for a potential disposal cell represents a special case. Instead of a rigid footing of small dimensions, the disposal cell would consist of a flexible soil embankment (clean-fill dike) several hundred feet across. In this case, foundation bearing capacity may be modeled as a failure of the embankment, where the soil underlying the base of the embankment fails. This mode of embankment failure is called base failure.

Bearing-capacity failure, or base failure, occurs usually as a shear failure of the soil supporting the foundation. The bearing capacity is, therefore, modeled using the shear strength parameters of the supporting soil, which are its apparent friction angle and apparent cohesion. Section 3.7.2.4 (Table 3.9) discusses the results of geotechnical laboratory unconsolidated-undrained triaxial compression tests for these shear strength parameters. Shear strength parameters were only measured for fine-grained soils in soil zones 1 and 4.

General shear failure is characterized by the existence of a well-defined failure pattern consisting of a continuous slip surface that initiates beneath the embankment, continues down into the ground, and exits back up at the ground surface. Because of the large size of the embankment for a potential disposal cell, the slip surface for a base failure may extend as deep as the Porters Creek Clay. However, no strength parameters were measured for the Porters Creek Clay due to sample disturbance or insufficient volume. For this reason, estimates of the shear strength parameters for the Porters Creek Clay (soil zone 6) were developed. The following considerations were made in selecting representative shear strength parameters for soil zone 6:

- Soil zone 6 is comprised of nearly 50% silt and 50% very fine sand. Consolidation tests show that the soil is compressible, demonstrating behavior typical of a fine-grained deposit. Results of index property tests show that zone 6 soils have a similar plasticity index to zone 4 soils. Therefore, zone 6 soils were modeled similar to zone 4 soils, with both friction and cohesion components.

- Estimates of N'_{60}, tip resistance, and unconfined shear strength from SPTs and SCPTs indicate that the field strength values for zone 6 soils are approximately double those for zone 4 soils.

- Based on empirical correlations to penetration resistance (Jumikis 1971; Peck, Hanson, and Thornburn 1974), zone 4 soils may be considered stiff clays with a cohesion between 1 and 2 tsf. Actual measured cohesion averages 1.5 tsf. Using these same correlations, zone 6 soils may be considered very stiff clays with a cohesion between 3 and 4 tsf.

Based on these considerations, an apparent cohesion of 3 tsf was selected for use in modeling zone 6 soils. In addition, an apparent friction angle of 8 degrees was selected for modeling zone 6 soils, which is the average of zone 1 and zone 4 soils. Although selection of these shear parameters may involve large uncertainty, the impact of that uncertainty is low. Zone 6 soils are known to be much stiffer than overlying soils, so that the critical slip surface for base failure does not lie within soil zone 6.

Acceptable minimum safety factors for design of shallow foundations are established during detailed design. Selection should consider such factors as the degree of reliability of all other parameters that enter into the design, such as frequency of design loads, strength and deformation characteristics of the soil mass, serviceability, and expected life of the structure, and the probability and consequence of failure. General guidance (Winterkorn and Fang 1975) for a permanent structure where the design load is likely to occur often and the consequences of failure could be disastrous, is that a safety factor of 3.0 may be appropriate.

A simplified model of estimating the base failure of an embankment fill was taken from Winterkorn and Fang (1975). In this model, zone 6 soils are considered sufficiently stiff relative to zones 1 and 4 soils so that the critical slip surface occurs at the top of the Porters Creek Clay. Two embankment-loading cases were analyzed:

- Case 1: A perimeter embankment slope of (6:1), 102 ft high, with an average unit weight of 115 pcf. In this case, the critical load at bearing-capacity failure is calculated to be 30.45 tsf. The actual maximum load is calculated to be 5.87 tsf. Therefore, the safety factor, with respect to bearing-capacity failure, would be $30.45 \div 5.87 = 5.2$. This is an acceptable margin of safety for bearing capacity.

- Case 2: An interior embankment slope of (3:1), 40 ft high, weighing 115 pcf. In this case, the critical load is 8.76 tsf and the actual maximum load is 2.3 tsf. Therefore, the safety factor with respect to bearing-capacity failure would be $8.76 \div 2.3 = 3.8$. This is an acceptable margin of safety for bearing capacity.

A numerical simulation model was also used to calculate base failure beneath the embankment. Only a static analysis was performed in this model since the liquefaction potential discussed in Chap. 5 is low at Site 3A. The analysis was performed using GSTABL7, a 2-dimensional limit equilibrium slope stability program that analyzes the embankment by a method of slices. The computer model generates hypothetical slip surfaces through the subsurface soil zones, divides the slip blocks into vertical slices, then calculates the equilibrium of each slice. A mathematical safety factor is then calculated for each slip surface as the sum of the moments resisting failure divided by the sum of the moments tending to cause failure (Winterkorn and Fang 1975). The most critical slip surface of all those analyzed by the computer model is the one resulting in the minimum calculated safety factor.

Acceptable minimum safety factors for design of embankment foundations are established during detailed design. Relatively small safety factors have usually been used in analyses of embankment slopes for their stability. Conventional guidance (Jumikis 1971) for an embankment structure is that a safety factor of 1.5 may be appropriate for deep-seated or rotational failure.

The following embankment foundation failure cases were analyzed. In all cases, a perimeter embankment slope of 6:1, 102 ft high, with an average unit weight of 115 pcf was assumed.

- Case 1: The strength of the embankment materials was disregarded so that base failure alone would be analyzed. This is very conservative. The critical slip surface was found to be a deep-seated rotational failure that occurred within the soil zone 1 loess layer. The minimum safety factor with respect to embankment base failure was calculated to be 1.69. This is an acceptable margin of safety for embankment base failure.

- Case 2: The strength of the embankment materials was again disregarded so that base failure could be analyzed. This is again very conservative. The critical slip surface was constrained so that a deep-seated rotational failure would be forced to occur within soil zone 4. The minimum safety factor with respect to embankment failure in this case was calculated to be 2.51. This is an acceptable margin of safety for embankment base failure.

- Case 3: The strength of the embankment materials was set at the very minimum required to achieve internal stability of the above-ground slope. This strength corresponds to an apparent friction angle of 9.5° and no cohesion. This is conservative since actual embankment materials would be designed using materials of higher strength. The critical slip surface was found to be a deep-seated rotational failure that occurred within soil zone 1. The minimum safety factor was calculated to be 2.42. This is an acceptable margin of safety for embankment base failure.

The results of the bearing-capacity analysis indicate that the bearing capacity of the foundation soils is adequate to support a potential disposal cell embankment at Site 3A.

4.5 SUMMARY

Geotechnical design considerations regarding fate and transport, settlement, and bearing capacity of a potential on-site CERCLA waste disposal facility have been discussed in this chapter, which could be used in any subsequent evaluation of the technical feasibility and effectiveness of such a facility.

The fate and transport model identifies the primary contaminant release and migration mechanisms from a filled disposal cell. Rainwater would infiltrate through the cover materials, react with the encapsulated waste, and produce leachate. Although leachate would be removed by the leachate collection system, it is assumed that the leachate collection system no longer functions and that the leachate would migrate downward to the water table and into the Terrace Deposits Flow System. The Terrace Deposits Flow System discharges to the UCRS beneath the plant and the UCRS in turn recharges the RGA. The primary receptor locations for Site 3A would ultimately be at the DOE property boundary and the Ohio River, although other receptors may be appropriate depending on specifically applicable regulations if an on-site CERCLA waste disposal facility were to be constructed.

The settlement model estimates the total settlement of a filled disposal cell. Calculations have shown that fill constructed to a height of 102 ft above the ground surface would result in more than 5 ft of settlement in the center of the disposal cell area. Differential settlement could be as high as 2 to 3 ft across the disposal cell. Settlement would occur relatively rapidly, with 90% of the settlement occurring in less than 2 years of fill placement.

The bearing capacity model estimates the ability of the soils beneath a filled disposal cell to support the weight of the fill. Calculations have shown that the bearing capacity of the foundation soils is adequate to support a potential CERCLA waste disposal facility at Site 3A.

5. EVALUATION OF LIQUEFACTION

As discussed in Chap. 1, the Project Core Team developed a list of seven questions that, when answered, would address seismic issues related to the siting of a potential CERCLA waste disposal facility at PGDP. This chapter reports on studies designed to address the following three questions:

- Question 1: Is there evidence of paleoliquefaction at or near the PGDP?
- Question 2: Is there paleoseismic evidence of local strong ground motion?
- Question 3: Is there potential for future liquefaction at Site 3A?

Table 5.1 repeats these three questions and presents a summary of the answers developed during the seismic investigation. Data, information, and details used to develop the responses presented in Table 5.1 are included in the body of this chapter.

Table 5.1. Summary of answers to Questions 1, 2, and 3 posed by the Project Core Team

Question	General answer
1. Is there evidence of paleoliquefaction at or near PGDP?	Field observations made along the Ohio River in the vicinity of PGDP found no large liquefaction features. Smaller scale paleoliquefaction features may have been present but remained unobserved due to their relatively small size or veneer of river deposits and vegetative cover. There is no definitive evidence of paleoliquefaction at PGDP based on results of field investigations conducted along portions of Bayou and Little Bayou Creeks. The literature does report some small liquefaction features located along the banks of the Ohio River, about 8 miles northeast of PGDP, and along the Post Creek Cutoff, about 12 miles northwest of PGDP.
2. Is there paleoseismic evidence of local strong ground motion?	The absence of large paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. The small liquefaction features that have been reported in the literature are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.
3. Is there potential for future liquefaction at Site 3A?	Many of the soils present at the site are clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet the criteria that distinguish those fine-grained soils that could experience large-scale strain, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. Some liquefaction within the sands and deformation within the silts and clays could occur at PGAs approaching 0.5 g.

The Paleoliquefaction Study and the Geotechnical Study performed as part of the Seismic Investigation program for Site 3A provided data used to develop the responses. These studies included the following pertinent activities:

- Review of historical information to identify areas that liquefied during past large earthquakes.
- Search for evidence of prehistoric (paleoliquefaction) features in the region.
- Evaluation to see if there is evidence of past liquefaction in soil core samples collected from Site 3A.
- Collection and evaluation of in-situ soil properties at Site 3A to assess liquefaction potential of site soils.

Each of these aspects of the Site 3A Seismic Investigation is described in the subsequent sections. To provide the proper context for these discussions, a brief background on seismically-induced liquefaction and liquefaction susceptibility precedes them.

5.1 BACKGROUND ON LIQUEFACTION

Liquefaction is a process by which saturated granular soils temporarily lose strength and act as a viscous liquid rather than a solid. To understand this phenomenon, it is important to review what happens to saturated soils during an earthquake.

Before an earthquake, individual soil grains are in contact with a number of the adjacent soil grains. The weight of the overlying soils forces contact between the grains, holding the individual grains in place and giving the soil its strength. Unless the soil grains are densely packed, there are still spaces between the grains. If the soils lie below the water table, these spaces are filled with water. These waters are called "pore waters," because they fill the "pore" spaces between the grains.

During an earthquake, seismic waves passing through saturated soils can cause loosely packed grains to become more densely packed. The reduction in overall volume "squeezes" the pore water, increasing the pore-water pressure between the soil grains. If the local conditions are such that drainage cannot quickly occur, the pore-water pressure may rise to a level approaching the weight of the overlying soil, reducing the contact between the individual grains. When this happens, the soil temporarily behaves as a viscous liquid rather than a solid, which is the condition known as liquefaction.

Because the available pore spaces are greatest in loose, poorly graded sand, these types of soil are most susceptible to liquefaction. As the available pore spaces decrease, the liquefaction potential also decreases. For example, in silty sands the available pore spaces between sand grains are partially filled with silt, reducing the amount of space for pore water. Consequently, these types of sands generally have less pore water available to "lift" the grains during an earthquake.

In clayey sands, the available pore water is reduced even more, resulting in an even lower potential for liquefaction. The space between grains also tends to reduce with geologic time, as the overlying weight compresses the soils. In addition, as soils age, the contacts between the individual grains may become "cemented" because of precipitation of minerals out of the local pore waters or weathering of the minerals that make up the grains.

Based on these factors, saturated Holocene-age to late Pleistocene-age sands are considered to be the types of soil most susceptible to liquefaction. Liquefaction has been observed, but is much less common, in older sands and saturated gravels and silty sands. Finer grained non-granular soils are not prone to liquefaction. However, under some very specific and limited conditions, fine-grained soils can experience large-scale deformation similar to liquefaction (Seed and Idriss 1982).

When earthquake-induced liquefaction occurs, several types of consequences can happen in the liquefied deposits, such as settlement of the ground surface, slope failure, or sand ejecting from fissures. When sand is ejected from fissures to the ground surface, distinct liquefaction features are formed. These liquefaction features are typically seen as sand blows, sand dikes, and sand sills. These are the types of liquefaction features that are of interest in this study.

The size of the liquefaction features that form during an earthquake depends on several factors, including the magnitude of the earthquake, the intensity of ground motions, and distance to the earthquake's epicenter. A site having saturated sands that is located near an earthquake epicenter and is subject to strong ground motions would exhibit large liquefaction features. Conversely, the same site located far from the epicenter or subject to only weak ground motions may exhibit small or no liquefaction features. Therefore, because Question 2 is asking for evidence of local strong ground motion, only large liquefaction features are of interest in this study.

5.2 HISTORICAL LIQUEFACTION STUDIES

Historical records of earthquake events include reports of liquefaction observations made following an earthquake. The following paragraphs discuss the available historical records for the central U.S., which include observations from two historical events.

During the winter of 1811 and 1812, a series of earthquakes occurred in the NMSZ. The first magnitude 7+ earthquake occurred on December 16, 1811. Two other magnitude 7+ shocks, one on January 23, 1812, and the other on February 7, 1812, followed the initial earthquake. The three main shocks probably reached intensity XII, the maximum on the Modified Mercalli scale. Although the magnitude of these earthquakes have been historically reported between 7.8 and 8.1, recent studies indicate magnitudes of approximately 7.5 likely occurred (Hough et. al. 2000).

Although the precise epicentral coordinates of the earthquakes are not known, accounts of the events suggest that the epicenter of the first earthquake (December 16) was probably in northeast Arkansas. The second main shock (January 23) likely occurred in Southeastern Missouri. The epicenter of the third shock (February 7) occurred near the town of New Madrid, Missouri, about 50 miles southwest of PGDP.

In the Mississippi and Ohio River valleys, landslides occurred during the 1811–1812 earthquakes along the steeper bluffs. As a result, the high banks caved and collapsed into the river. Most pertinent to the discussions here was the observation that large areas subsided because of liquefaction and were covered with water and sands that had emerged through fissures. The region extending from Cairo, Illinois, to Memphis, Tennessee, and from Crowleys Ridge to Chickasaw Bluffs, Tennessee, was characterized by sand ejections, fissuring, severe landslides, and caving of stream banks. Closer to PGDP, liquefaction was reported along the Ohio River at Fort Massac, Illinois (Hough, et. al. 2000).

Figure 5.1 presents information taken and modified from a recent USGS open file report published on the World Wide Web at http://pubs.usgs.gov/openfile/of98-488 (USGS 2002b). Figure 5.1a provides an overview of the location of the NMSZ and also shows epicenters and major areas of liquefaction from the 1811–1812 earthquakes. Much of the epicentral area of these earthquakes is very favorable for the generation of liquefaction, because geologically young, clean, saturated sands are present. As discussed above, these types of granular soils are especially susceptible to liquefaction during an earthquake. Figure 5.1b is a photograph taken in the epicentral area of a past earthquake and shows that the sands ejected during those earthquakes are still visible from the air.

Available information suggests that no liquefaction or ground failure occurred in the upland surface at the present PGDP during the 1811–1812 earthquakes (USGS 1994). The only reported failures in the upland surface during these earthquakes were slope failures on bluffs of the Mississippi River, significantly closer to the NMSZ and the 1811–1812 epicenters than PGDP.

The only other historical earthquake known to have caused liquefaction in the region occurred on October 31, 1895. This magnitude 6.7 earthquake occurred near Charleston, Missouri. This event is the largest earthquake to occur in the area since the 1811–1812 series. More limited liquefaction was associated with this event. Sand blows were observed in an area southwest of Charleston, Missouri, and south of Bertrand, Missouri. Isolated occurrences of sand blows also were reported north and south of Charleston (Stover and Coffman 1993). The reported liquefaction occurred in low-lying areas where geologically young sands are present.

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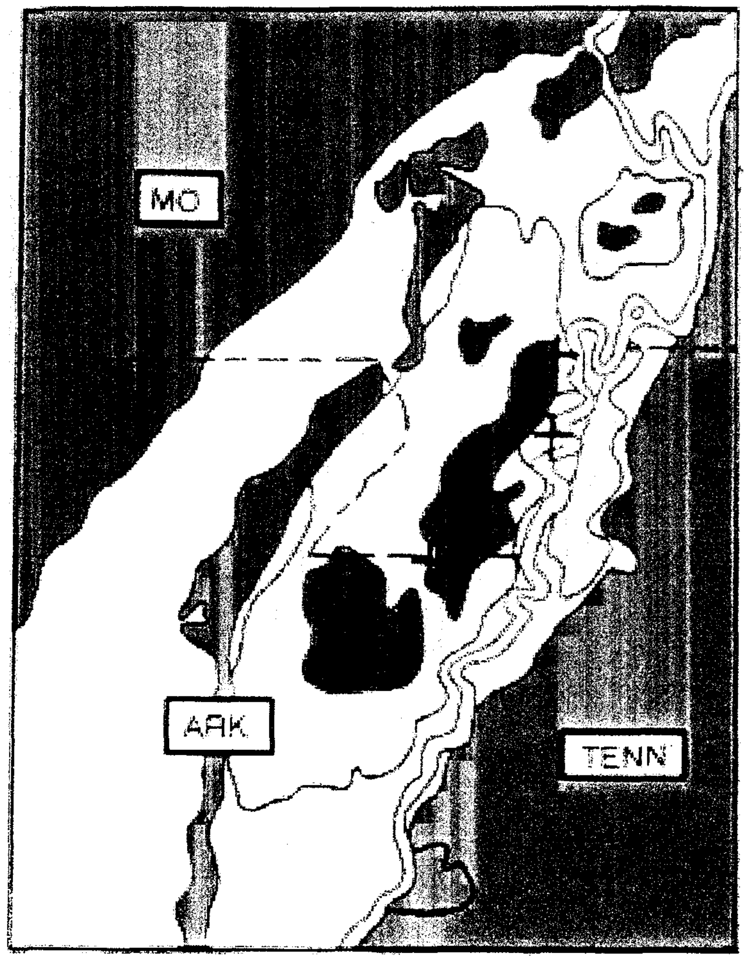
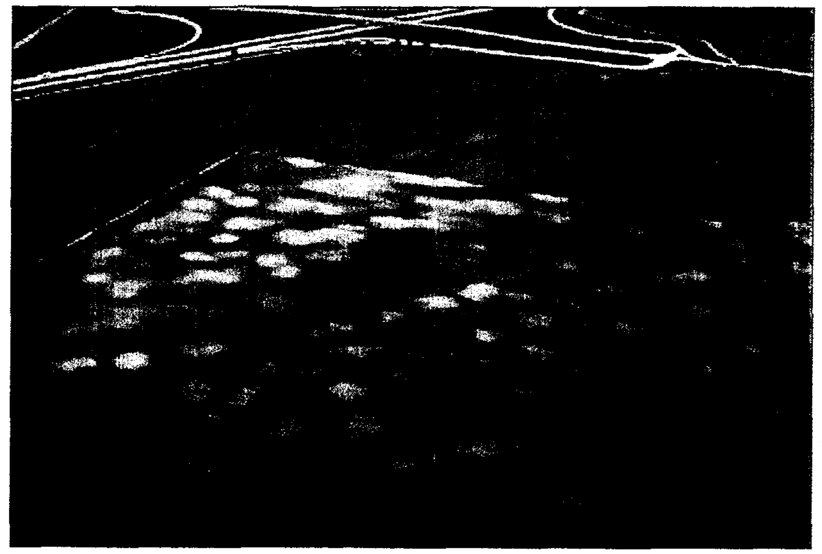
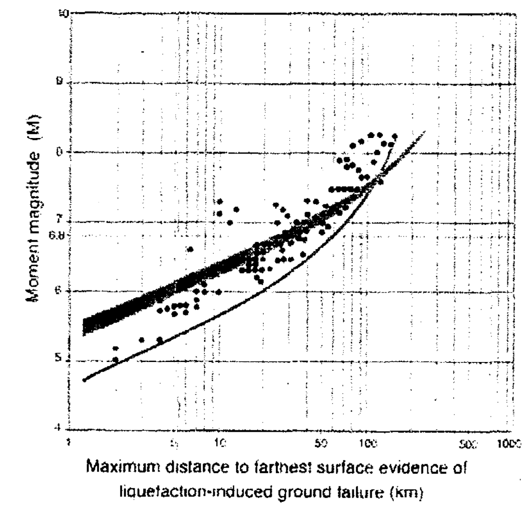


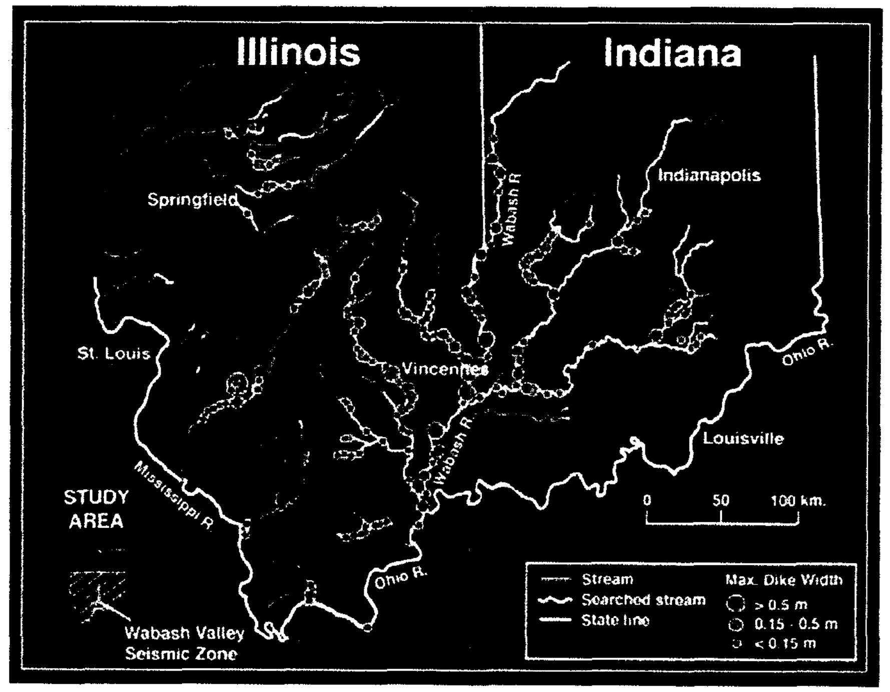
(a). **Distribution of 1811-1812 Liquefaction features:** Crosses show the approximate epicenters of the three largest earthquakes. The blue area indicates uplands where little or no liquefaction occurred during the 1811–1812 earthquakes. The white areas are recent alluvium. In these areas, some regions had sands cover as much as 25% of the land surface (orange). In other areas, sands covered approximately 1% of the land surface (yellow).



(b). **Aerial View of 1811–1812 Liquefaction Features:** The light circular features are vented sands brought up from underlying sands. Note the tree in center for scale. A search was implemented for similar expressions in aerial photographs of the PGDP area.



(c). **Earthquake Magnitude vs. Distribution of Liquefaction:** The thin green curve is based on worldwide data (Ambraseys 1988). The shaded orange curve is based on data from the central U.S. (Obermeier 1993–Pond 1996). Modified from Obermeier 1999.



(d). **Distribution of Paleoliquefaction Features in Illinois and Indiana:** Previous researchers have searched hundreds of kilometers of stream and riverbanks in southern Illinois and Indiana. Red dots show locations of paleoliquefaction features. The size of the dot indicates the size of the widest sand dike at that site. Features are more common and larger in the general vicinity of Vincennes. Note that the only features found on the Ohio River near PGDP are at Ft. Massac, Illinois. This was the site of liquefaction during the 1811–1812 New Madrid earthquake.

(e). **Example of Large Paleoliquefaction Feature:** This feature was discovered in the Wabash Valley seismic zone during reconnaissance by boat. Sand and gravel ejected onto the ground surface during an earthquake approximately 6100 years ago were subsequently covered by several feet of overbank silt and clay.

Note: All figures are modified from Obermeier (1999) (USGS 20026).

Fig. 5.1. Examples of liquefaction features in the PGDP region.

U.S. DEPARTMENT OF ENERGY
DOE OAK RIDGE OPERATIONS
PADUCAH GASEOUS DIFUSION PLANT

BECHTEL JACOBS COMPANY, LLC
MANAGED FOR THE US DEPARTMENT OF ENERGY UNDER US GOVERNMENT CONTRACT DE-AC-05-98OR22700
OAK RIDGE, TENNESSEE • PADUCAH, KENTUCKY • PORTSMOUTH, OHIO

Science Applications International Corporation
P.O. Box 2505
Oak Ridge, Tennessee 37831

Figure No. 5.1
DATE 7/26/02

5.3 PALEOLIQUEFACTION STUDIES

The historic earthquake record for the central U.S. covers only a few hundred years. Paleoliquefaction studies search for evidence of prehistoric earthquakes and, thereby, offer a way to extend the earthquake record back hundreds or even thousands of years. This information can help to better estimate the long-term hazard in terms of earthquake magnitude or severity of shaking. The estimated size of each prehistoric earthquake can be determined by several factors. First, only earthquakes larger than magnitude 5.0 to 5.5 are capable of producing widespread liquefaction (Ambraseys 1988). Second, the areal extent of liquefaction correlates with the intensity of ground motions. The stronger the earthquake, the more widespread and stronger the shaking. Figure 5.1c illustrates this relation based on worldwide empirical data and presents information developed for the central U.S. As shown, a magnitude 7.0 earthquake is predicted to generate liquefaction features in loose sands out to about 30 miles from the epicenter. A magnitude 7.5 earthquake is predicted to generate liquefaction in loose sands out to about 70 miles. Previous searches for evidence of liquefaction have been conducted in the NMSZ to the southwest of PGDP and in the WVSZ to the northeast of PGDP. Although those studies did not focus on the Site 3A vicinity, they did extend into the general area.

5.3.1 Past Paleoliquefaction Studies in NMSZ

Field studies published in 1912 by M. L. Fuller of the USGS, provided topographic and geological evidence of large magnitude earthquakes in the NMSZ predating the 1811–1812 sequence. This evidence included ground cracks as large as those caused by the 1811–1812 earthquakes in which trees at least 200 years old had grown, suggesting that the old cracks at were at least 200 years old. Fuller also found indications of pre-1811 sand dikes, suggesting that the pre-1811 earthquakes generated liquefaction. These observations indicate that large earthquakes similar to the 1811–1812 events occurred in the recent geologic past.

More recently, detailed searches for paleoliquefaction features have been conducted throughout the NMSZ (Tuttle and Schweig 1997). These investigations found numerous paleoliquefaction features in the epicentral area of the 1811–1812 earthquakes. Radiocarbon dating studies indicate earlier earthquake sequences, similar in location and size to the 1811–1812 sequence, occurred in the past several thousand years.

Tuttle and Schweig (1997) interpret these features to indicate that at least two, possibly three, large to great earthquakes or earthquake sequences occurred in the region between 250 and 1150 years ago. One probably occurred about 1100 ± 100 years ago, a second event probably occurred about 700 ± 100 years ago, and a third event may have occurred about 400 ± 100 years ago. In addition, there is evidence for at least three large events between 1220 and 5340 years ago.

5.3.2 Past Paleoliquefaction Studies in WVSZ

A detailed search for additional evidence of paleoliquefaction features was conducted in WVSZ northeast of PGDP. Several investigators have searched for paleoliquefaction evidence of prehistoric earthquakes similar to the 1811–1812 events (McNulty and Obermeier 1997; Munson et. al. 1997; Obermeier 1998). Field investigations in southern Illinois and southern Indiana included the banks of the Ohio River from approximately 15 miles west of the PGDP to the river's confluence with the Wabash River, approximately 60 miles northeast of PGDP (REI 1999).

These investigations extend into the PGDP area and further to the north and northeast into Indiana and Illinois. Figure 5.1d shows the location of liquefaction features identified during these investigations. A local search that was focused along the Ohio River adjacent to PGDP found no evidence of paleoliquefaction (Obermeier 1998). The closest liquefaction features to PGDP are located along the banks of the Ohio River, about eight miles to the northeast. These features are small and located in the general vicinity of Fort Massac, Illinois, a location where liquefaction was reported during the February 7, 1812, earthquake. Obermeier (2002) notes that these features were small and relatively unweathered, suggesting that they were probably outlying liquefaction features resulting from the 1811–1812 sequence. The banks of the Ohio River yielded no evidence of a paleoliquefaction episode associated with a large earthquake originating in the vicinity of PGDP. Small dikes have been reported along the Post Creek Cutoff, about 12 miles northwest of PGDP. Obermeier concluded that seismic shaking in the vicinity of Metropolis, Illinois, and Paducah, Kentucky, has not exceeded Modified Mercalli intensity VIII–IX in Holocene times.

Further to the north and northeast, investigations along riverbanks found extensive evidence of paleoliquefaction. Paleoliquefaction studies have been conducted along most of the major rivers of southern Indiana and Illinois. In these studies, riverbank sediments are studied for evidence of earthquake-induced sand dikes, sand sills, and sand blows. Figure 5.1e illustrates how evidence of paleoliquefaction can be identified from a boat survey.

Based on these paleoliquefaction studies, investigators have recognized at least eight prehistoric earthquakes within the last 20,000 years that were strong enough to cause liquefaction (REI 1999). These earthquakes occurred outside of the NMSZ, but did not originate in the PGDP area. Of these eight earthquakes, six were probably of magnitude greater than 6 and at least two are estimated to have exceeded magnitude 7 (Obermeier 1998).

The largest prehistoric earthquake in the paleoliquefaction record occurred about 15 miles west of Vincennes, Indiana. This earthquake is estimated to have occurred 6100 ± 200 years ago. Based on the size of sand dikes and distribution of liquefaction features, a magnitude of 7.5 has been estimated for the prehistoric event. The epicenter would lie approximately 120 miles northeast of PGDP. In addition, evidence of paleoliquefaction was found along the Sangamon, White, and Kaskaskia rivers.

5.3.3 Paleoliquefaction Study for this Seismic Investigation

A new search for paleoliquefaction features was initiated as part of this Seismic Investigation. This study focused on the area within approximately 15 miles of PGDP with the purpose of determining the following:

- the extent of Holocene-age liquefaction in the PGDP region, and
- the extent and source area of paleoliquefaction features, if found.

Current seismic risk and ground motion models for PGDP are based on seismic sources that result from distant New Madrid-type earthquakes. The objective of the Paleoliquefaction Study was to confirm whether Holocene-aged paleoliquefaction features in the vicinity of PGDP had resulted from such distant earthquakes or from a local earthquake source. If paleoliquefaction features were found to have resulted from a local source, then current seismic risk and ground motion models would need to be re-evaluated.

A detailed discussion of the search for paleoliquefaction features at and near PGDP is presented in Appendix A. The Paleoliquefaction Study included the following activities:

- Identification of areas where conditions were potentially favorable for the generation and preservation of paleoliquefaction features.

- Review of aerial photographs for these areas for evidence of liquefaction features.

- Reconnaissance-level field investigations to refine the list of potential field areas.

- Ground inspection of high priority areas.

Using the information presented in Sects. 5.1 through 5.3 as a guide, the first task in the review of existing data was to identify areas where conditions are potentially favorable for the generation and preservation of paleoliquefaction features. This aspect of the study found that the primary area of interest for the paleoliquefaction study lies within the floodplains of the Ohio River and its major tributaries, primarily Mayfield Creek, and within the basin of the ancestral Cache River (located in southern Illinois).

Next, available aerial photographs and county soil surveys of the U.S. Soil Conservation Service were viewed to identify areas of sandy or gravelly soils series and photographic evidence of paleoliquefaction features like those presented in Fig. 5.1b. The analysis of the aerial photography did not distinguish any paleoliquefaction features; however, it was found that the soils series maps for some Illinois counties did identify discrete features called "sand spots." Several Illinois sites with sand spots were selected for consideration as field study sites.

Subsequent field reconnaissance was conducted across the region to refine the potential list of candidate field locations. Those field sites where there appeared to be the best potential for the presence and observation of paleoliquefaction features and better accessibility were carried forward for further consideration.

Before ground inspections could begin, property owners had to be contacted to secure access agreements. Project personnel reviewed property records in the respective county courthouses to identify property owners to be contacted to obtain Right of Entry for the Paleoliquefaction Study. While this activity was ongoing, field studies were implemented in areas where access requirements were not an issue. This included an evaluation of the banks of the Ohio River via boat and ground investigations along portions of Bayou and Little Bayou Creeks that are located on DOE property.

Ohio River Bank Survey: Project personnel conducted a boat survey of the banks of the Ohio River across the entire width of the region of study and beyond in a search for paleoliquefaction features (Fig. 5.2). The Ohio River, via boat, provided relatively easy access to exposures of the region's sand and gravel deposits to further assess the candidate field study sites and other areas of interest. Because no Right of Entry had been granted at the time of the boat survey, project personnel were limited to visual inspections and documenting existing conditions of the bank exposures. No samples for ^{14}C age dating were collected. The riverbank survey covered 50 miles of the Ohio River and included both banks. The goals of this survey follow:

- Assess the general suitability of Ohio River bank sediments for the Paleoliquefaction Study (age of exposed sediments and sequence of soil textures).

- Locate syndepositional carbonaceous material (e.g., buried logs) for future collection of ^{14}C samples.

- Identify large-scale paleoliquefaction features, if they exist.

- Find promising areas for closer-look ground inspections.

This survey of the banks of the Ohio River provided a valuable regional perspective that could not be duplicated by any other data set. The Ohio River bank survey resulted in descriptions of 55 outcrops, which are presented in Appendix A. Photodocumentation of the riverbanks in the form of digital photographs and camcorder tapes are available in project files.

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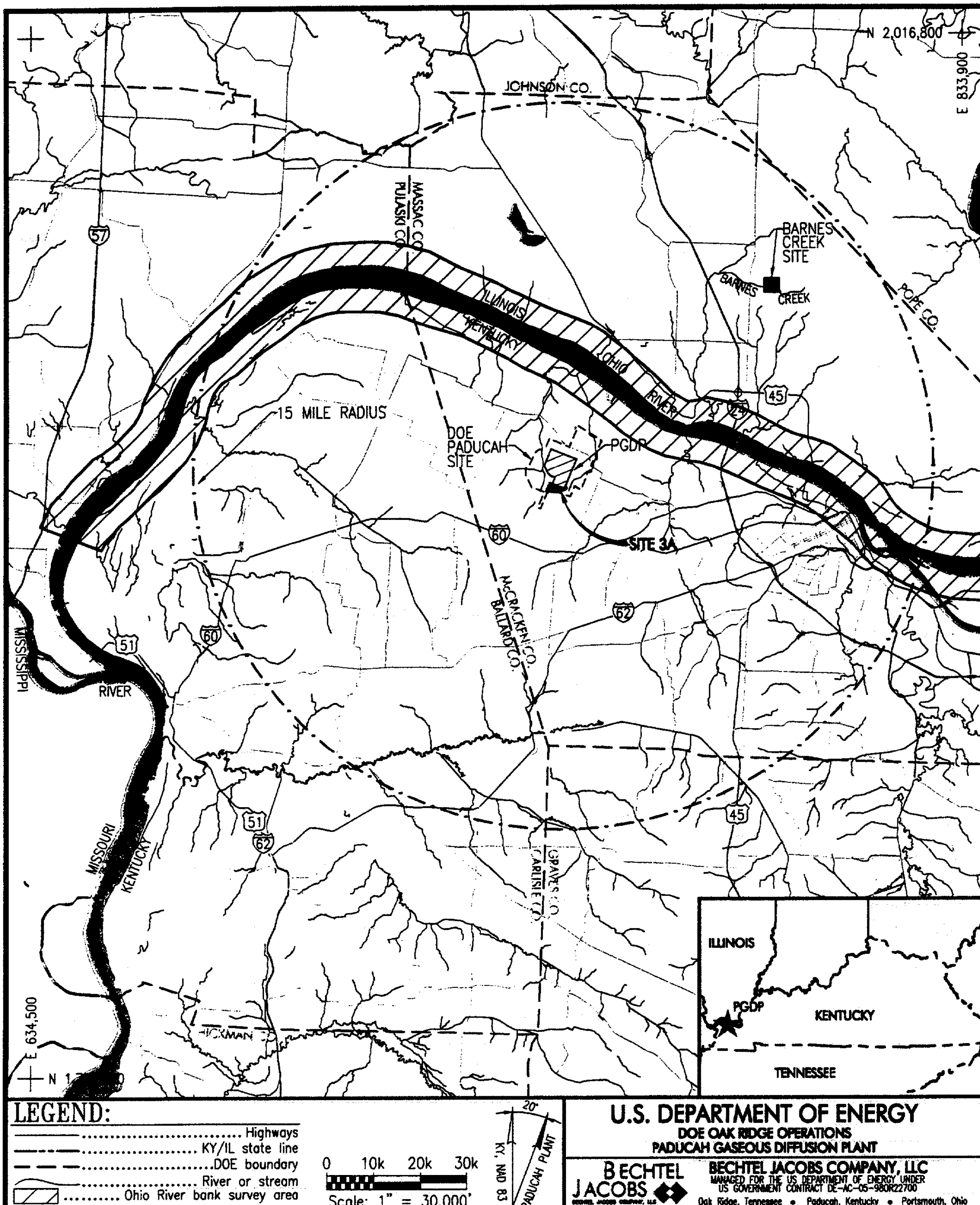


Fig. 5.2. Area of the Ohio River bank survey.

Figure No. /99049/DWGS/P34ORBS

DATE 07-09-02

No obvious paleoliquefaction features were observed along the Ohio River in the vicinity of PGDP. The riverbank afforded adequate exposure of the sediments such that large paleoliquefaction features, as shown in Fig. 5.1e, would have been obvious. Smaller-scale paleoliquefaction features, such as the ones from the 1811–1812 events reported by previous investigators near Fort Massac, Illinois, may have been present but were not observed because of their relatively small size or the typical veneer of river deposits and vegetative cover.

The absence of clear evidence of paleoliquefaction along this portion of the Ohio River should be viewed with the following caveats in mind:

- A significant portion of the Ohio River bank deposits appears to be of too recent origin (evidenced by recent cultural debris) to have experienced prehistoric earthquakes.
- Some of the riverbank deposits are silt and clay units containing few sand lenses capable of producing paleoliquefaction features.
- Smaller-scale paleoliquefaction features may have been present but could not be observed because of their relatively small size or the typical veneer of river deposits and vegetative cover.
- Some paloeliquefaction features may have existed in the past but could have been washed away by subsequent floodwaters.

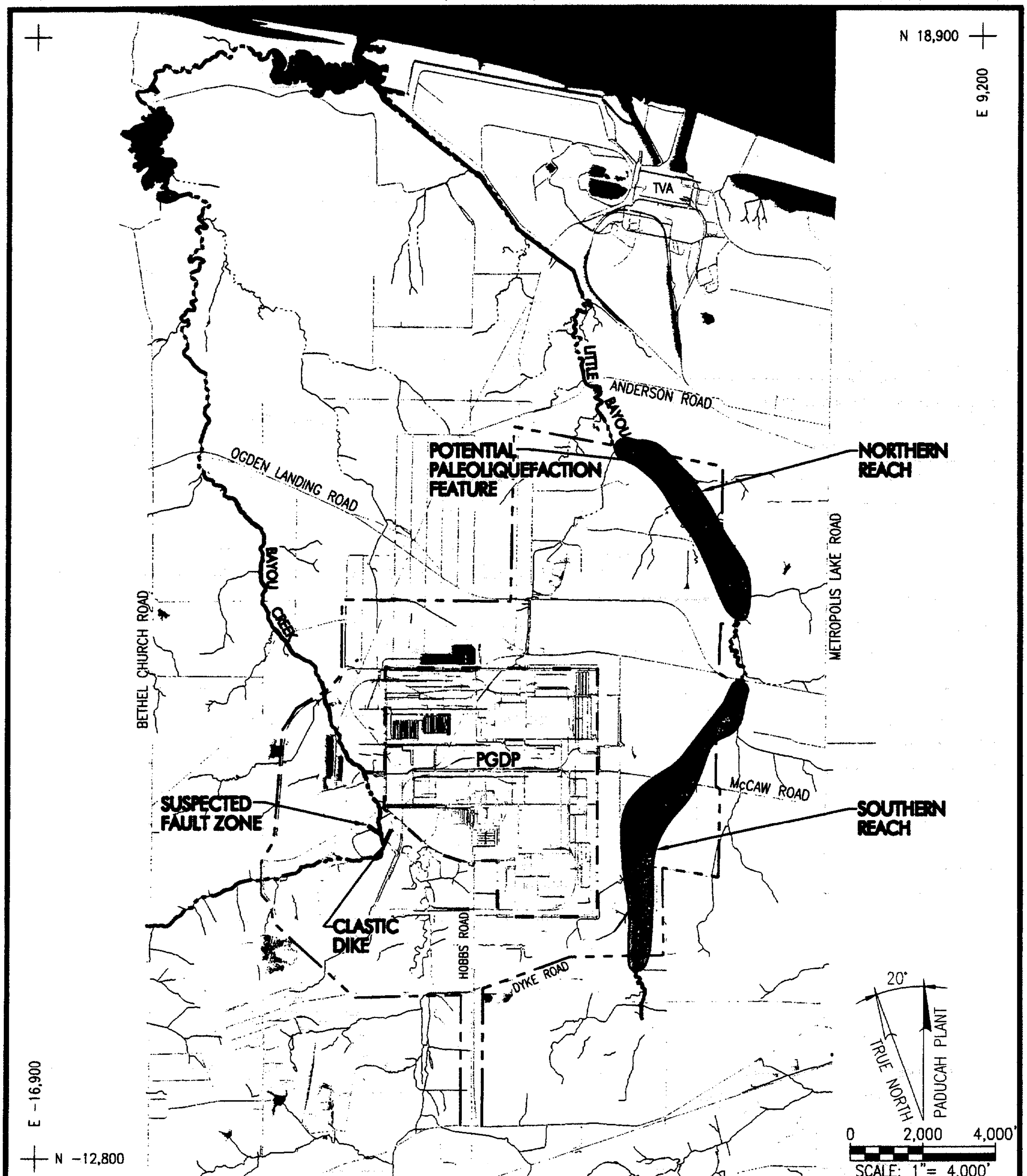
This study did identify suitable field sites for more detailed study of the riverbank sediments. The follow-up investigations at these sites were not completed because of delays in obtaining access. In addition, the results of the regional and site-specific Fault Studies suggested that further searching for paleoliquefaction features was not warranted.

Field Investigation along Barnes Creek: The regional Fault Study conducted at the Barnes Creek site included a survey of faulting and related structures in the exposed banks of a 2600-ft reach of the creek. This creek bank survey found no clear evidence of paleoliquefaction features. Investigators noted that characteristics of the geologic units were unfavorable to the formation of paleoliquefaction structures. A review of soil cores collected during DPT sampling at Barnes Creek also yielded no evidence of liquefaction.

Field Investigations along Bayou and Little Bayou Creeks: Bayou and Little Bayou Creeks are Ohio River tributaries that border the industrial area of PGDP on the west and east sides, respectively. Bayou Creek begins south of the PGDP and flows across the north-to-south width of the West Kentucky Wildlife Management Area (WKWMA). Little Bayou Creek begins south of the PGDP, within the WKWMA, and transects the majority of the north-to-south width of the WKWMA (Fig. 5.3).

The field investigation of Bayou Creek proceeded downstream (south to north). A reported sand dike ("clastic dike") mapped in the bed of Bayou Creek near the southwest corner of the PGDP industrial area (USGS 1966) is of potential interest. This clastic dike crosscuts the Paleocene Porters Creek Clay and likely predates the period of interest to this Seismic Investigation, which is the Holocene Epoch.

Overall, Bayou Creek afforded frequent exposure of the creek bank sediments. Over the 8000 ft surveyed on Bayou Creek, the average distance between documented exposures was approximately 250 ft. Most exposures of the creek bank sediments, along the entire reach of Bayou Creek that is located on DOE property, consisted of iron-cemented gravel. These gravel deposits in the south half of the creek surveyed are Pliocene/Pleistocene-age gravels of the Terrace Deposits, as discussed in Chap. 3.



LEGEND:

- ....DOE property boundary
- ....PGDP boundary
- ....Asphalt road
- ....Stream

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DOE OAK RIDGE OPERATIONS
PADUCAH GASEOUS DIFFUSION PLANT

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BECHTEL JACOBS COMPANY, LLC
MANAGED FOR THE US DEPARTMENT OF ENERGY UNDER US GOVERNMENT CONTRACT DE-AC-05-98OR22700
Oak Ridge, Tennessee • Paducah, Kentucky • Portsmouth, Ohio

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P.O. Box 2502
Oak Ridge, Tennessee 37831

Fig. 5.3. Location of Bayou and Little Bayou Creeks.

Figure No. /99049/DWGS/P34SK536
DATE 07-02-02

Similar gravel exposures found in the north half of this survey likely belong to the Pleistocene-age Upper Continental Deposits. Clay or silt underlies the gravel units to the north. The creek bed was generally covered by loose gravel to the south.

Where overlying units are uncovered, a massive silt typically covers the gravel. This silt is thought to be a loess deposit of Pleistocene age. Occasional exposures of apparent Holocene-age creek deposits occur along the creek. There were very few locations where organic material was observed within the sediments that could be sampled to ^{14}C age date the units.

The walkdown of Bayou Creek on DOE property did not reveal any obvious paleoliquefaction features; however, a suspect fault zone (which may be only a creek bank slump feature) was noted in the west creek bank near the southwest corner of the PGDP industrial area.

The walkdown of Little Bayou Creek proceeded in a general downstream direction over two reaches, as permitted by access points (south to north). The south reach of Little Bayou Creek (approximately 9000 ft long) is heavily vegetated with very few exposures of creek bank sediments. Exposures of creek bank sediments are comparatively frequent along the north reach of Little Bayou Creek (approximately 7000 ft long), averaging a documented exposure every 450 ft. Along the north reach, the exposures typically are 20 to 50 ft long and 4 to 6 ft high.

Typical creek bank exposures in Little Bayou Creek, over both the south and north reaches, consist of massive silt overlying clay with the bed of the creek eroded into the clay member. These units are thought to consist of Upper Continental Deposits and loess, both Pleistocene in age. Little Bayou Creek appears to have more exposures of younger creek deposits than Bayou Creek. Like Bayou Creek, however, there are few areas where samples could be collected to ^{14}C age date the sediments.

Faulting was not observed in any outcrops in Little Bayou Creek. One questionable potential paleoliquefaction feature was identified. In this outcrop, located approximately 6000 ft north of PGDP, a sand-filled fracture crosscuts the clay unit that is exposed in the creek bed, and discrete sand lenses appear to be present in the overlying silt unit that is exposed in the creek bank. It could not be determined if the fracture is a desiccation crack infilled with overlying sand or if it actually represented paleoliquefaction.

Summary of Paleoliquefaction Study: The historical data review, Ohio River bank survey, and survey of soil exposures on Bayou and Little Bayou Creeks, accomplished the basic intent of the Paleoliquefaction Study. No clear evidence of paleoliquefaction associated with a local earthquake source was found. A large percentage of the available exposures of Holocene-age soils, across the 15-mile radius of the study area, were observed during the Ohio River bank survey.

These field observations found no large liquefaction features. The river bank afforded adequate exposure of the sediments such that if large liquefaction features were present, they would have been obvious. Smaller-scale paleoliquefaction features may have been present but were not observed because of their relatively small size or veneer of river deposits and vegetative cover. It should be noted that the available exposures may only provide a record for the late Holocene, because of the relatively young age of the deposits along the Ohio River.

The walkdown of Bayou and Little Bayou Creeks did not find definitive evidence of paleoliquefaction. The soils exposed in the creek banks are predominantly silts, clays, or cemented gravels that are not typically prone to liquefaction.

5.4 REVIEW OF SITE 3A SOIL CORES

Soil samples were collected from soil borings and DPT boreholes at Site 3A during the Geotechnical Study. These samples were examined for any visual evidence of liquefaction. Boring logs and soil cores collected at and in the immediate vicinity of Site 3A were reviewed by a geologist experienced in the identification of liquefaction features. The information reviewed follows:

- boring logs collected just northwest of Site 3A as part of preliminary characterization activities at a potential DUF_6 site,
- cores from the 400 ft deep soil boring (DB-02) drilled at Site 3A,
- samples collected in Site 3A SPT borings, and
- DPT cores collected at Site 3A.

No evidence of liquefaction (such as sand dikes or sand veins) was observed. It should be noted that, while no liquefaction features were observed, the data provided only limited information because sampling could be done only at discrete vertical borehole locations. Liquefaction features are frequently of a smaller size than the spacing between boreholes and, therefore, may not have been intersected by a particular borehole. Consequently, these findings should be viewed with this limitation in mind.

5.5 EVALUATION OF LIQUEFACTION POTENTIAL AT SITE 3A

A key aspect of the Geotechnical Study at Site 3A was the collection of in-situ data to characterize the potential for liquefaction. This included drilling SPT borings, collecting SCPT sounding data, and measuring shear-wave velocities. In addition, geotechnical laboratory data were collected for discrete soil samples to measure moisture content, grain size, and plasticity. In this section, these data are used to quantitatively evaluate the liquefaction potential for each of the six soil types encountered at Site 3A (which are described in Chap. 3).

This evaluation included three steps. First, the data were screened using general screening guidance (Lew 2001) to eliminate soil zones that consist of predominantly fine-grained soils, which are not prone to liquefaction. Second, a quantitative evaluation of liquefaction potential was made by comparing the in-situ soil strength data to an upper "bounding" value of the stress caused by an earthquake. This step was used to identify those soils at Site 3A that may be prone to liquefaction under a very large-magnitude earthquake and strong ground motions. Third, a quantitative evaluation of liquefaction potential was made by again comparing the soil strength to earthquake-induced stresses, this time using a range of ground motions. This step was used to identify the relative likelihood of liquefaction occurring in a given soil zone under different earthquake-induced ground motions.

The following subsections discuss each of the three evaluation steps.

5.5.1 Step 1: Application of General Screening Criteria

This evaluation step screens the soils encountered at Site 3A using general screening guidance to eliminate soil zones that are not prone to liquefaction. As stated in Sect. 5.1, the types of soils most susceptible to liquefaction are loose, saturated sands. General screening guidance (Lew 2001) suggests that liquefaction is not a credible concern in areas where the following subsurface conditions are present:

- ground-water levels are more than 50 ft bgs,
- "bedrock" or similar "lithified" materials are present, or
- soils have a clay content (particle size <0.005 mm) greater than 15%.

As presented in Chap. 3, six soil zones have been identified at Site 3A. The characteristics of these soil zones are presented in detail in Chap. 3. With the exception of the upper portions of soil zone 1, all of the soils lie below the water table. Consequently, the first screening criterion does not eliminate any of the Site 3A soil zones from further consideration. The second criterion eliminates the bedrock beneath Site 3A from consideration, but is not relevant for soils. Application of the third criterion suggests that soil zones 1, 2, and 4 are not prone to liquefaction. As shown on Table 5.2, the average clay fines content (<.005 mm) for each of these three soil zones is greater than 28%. Therefore, the high percentages of fines in these units greatly reduce their liquefaction potential.

However, based on empirical data (Wang 1979; Zhou 1981), Seed and Idriss (1982), have proposed an additional screening criterion. This criterion applies to fine-grained soils that may be susceptible to severe strength loss, similar to liquefaction, when an earthquake occurs. These are soils that meet each of the following characteristics:

- percent finer than 0.005 mm (0.000197 in.) less than 15%,
- liquid limit less than 35,
- water content greater than 0.9 × liquid limit, and
- liquidity index less than 0.75.

As shown in Table 5.2, soils in zones 1, 2, and 4 do not meet the characteristics required to meet this additional screening criterion. Therefore, it can be concluded that these soil zones are not prone to liquefaction nor are they susceptible to severe strength loss. Soil zones 1, 2, and 4 are eliminated from further consideration and are not carried forward into the quantitative evaluation of liquefaction potential. Conversely, soil zones 3, 5, and 6 are granular soils and have been carried forward for the quantitative evaluation.

5.5.2 Step 2: Application of Quantitative Bounding Criteria

The next step in this evaluation of the liquefaction potential at Site 3A requires the calculation of variables that express the demands placed on soils during an earthquake and the resistance of the soils to liquefaction. The earthquake demand is a measure of the stress induced by an earthquake on the soil and is expressed as the cyclic stress ratio (CSR). The resistance of the soils to liquefaction is a measure of the soil strength and is expressed as the cyclic resistance ratio (CRR).

Liquefaction occurs whenever the earthquake-induced stress exceeds the soil's strength (or resistance). Mathematically, the potential for liquefaction to occur is calculated by comparing the values of CSR vs. CRR. Where CSR exceeds the corresponding CRR (stress exceeds resistance), the potential for liquefaction is present. Conversely, where CRR exceeds CSR (resistance exceeds stress), liquefaction would not be expected.

Table 5.2. Average soil properties for fine-grained soil zones 1, 2, and 4 at Site 3A

Station	Sample no.	Depth (ft)	USCS symbol	Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Liquidity index (%)	Fines content <0.075 mm (%)	Clay fines content <.005 mm (%)
						Soil zone 1				
SB-01	3	4-6'	CL	27.6	31	19	12	0.72	97.9	35
SB-01	4	10-12'	CL	22.9	28	17	11	0.54	87.4	25
SB-02	3	4-6'	CL	24.6	32	20	12	0.38	97.7	31
SB-02	5	12-14'	CL	23.3	30	19	11	0.39	95.9	28
SB-03	4	6-8'	CL	23.7	31	20	11	0.34	97.1	24
SB-05	4	6-8'	CL	26.9	38	20	18	0.38	93.4	29
SB-06	3	6-8'	CL	29.2	35	20	15	0.61	97.7	26
SB-06	6	14-16'	CL	22	32	16	16	0.38	88	31
DB-02	3	8-10'	CL	18.7	29	20	9	-0.14	94.9	22
	Average			24.3 ± 3.2	31.8 ± 3.1	19.0 ± 1.5	12.8 ± 2.9	0.4 ± 0.2	94.4 ± 4.1	27.9 ± 4.1
						Soil zone 2				
SB-03	7	16-18'	CL	20.9	30	16	14	0.35	915	27
SB-06	10	22-24'	CL	13.3	28	13	15	0.02	60.1	32
DB-02	7	16-18'	CL	24.3	40	16	24	0.35	93.1	37
DB-02	11	24-26'	CL	21.7	32	14	18	0.43	65.6	34
	Average			20.1± 4.7	32.5 ± 5.3	14.8 ± 1.5	17.8 ± 4.5	0.3 ± 0.2	77.6 ± 17.2	32.5 ± 4.2
						Soil zone 4				
SB-01	11	24-26'	MH	64.4	102	53	50	0.23	93.5	50
SB-01	15	32-34'	MH	55.4	95	53	42	0.06	70.6	17
SB-02	15	32-34'	CL-ML	17.9	20	13	7	0.70	56.3	20
SB-03	17	36-38'	CL	30	43	21	22	0.41	92	41
SB-03	19	44-46'	MH	62.6	106	50	56	0.23	87.4	44
SB-05	14	30-32'	SC	23.8	39	14	25	0.39	26.3	13
SB-05	18	42-44'	SC	23.7	46	15	31	0.28	44.6	23
DB-02	14	30-32'	CL	19.9	25	12	13	0.61	58	31
DB-02	17	36-38'	CL	24.9	39	18	21	0.33	84.7	52
DB-02	21	48-50'	ML	24.7	44	NP	NP	NP	85.4	52
	Average			34.7 ± 18.4	55.9 ± 32.3	27.7 ± 18.5	29.7 ± 16.6	0.36 ± 0.20	69.9 ± 22.8	34.3 ± 15.3

Note: Liquidity index = (moisture content – plastic limit) ÷ (plasticity index)

CL = clean clay ML = inelastic silt

MH = elastic silt USCS = Unified Soil Classification System

It is generally agreed that CSR can be expressed by equation 5-1 (Youd and Idriss 1997):

$$CSR = 0.65(a_{max})\left(\sigma_v / \sigma_v'\right) r_d \tag{5-1}$$

where

a_{max} = peak horizontal ground acceleration in g,
σ_v = total vertical overburden stress in tsf,
σ_v' = effective vertical overburden stress in tsf, and
r_d = stress reduction factor (varies from 1 at ground surface to 0.5 at depths greater than 100 ft).

Values of CRR have been established from empirical correlations using extensive databases. Originally, the empirical correlations were derived for sites where corrected SPT values (N'_{60}-values) could be correlated with liquefied strata (Seed and Idriss 1971). Subsequently, similar empirical correlations were derived for CRR based on cone penetrometer resistance (Robertson and Wride 1997) and shear-wave velocities (Stokoe et al. 1988).

Seed and Idriss (1971, 1982) with refinements by Seed et al. (1983), Seed et al. (1985), Seed and De Alba (1986), and Seed and Harder (1990), establish procedures that compare CSR and CRR. This relationship is illustrated in Fig. 5.4. If the ratio of CSR/CRR lies to the left of the curve (Point A), liquefaction is possible. If the ratio of CSR/CRR lies to the right of the curve (Point B), liquefaction is not probable.

One result of the past empirical studies is the definition of CRR values that represent an upper bound above which liquefaction is not likely to occur, no matter what the corresponding CSR value. In general the screening criteria associated with these bounding values of CRR follow:

- SPT-derived CRR: Corrected N'_{60}-values greater than or equal to 30 blows per ft.
- SCPT-derived CRR: Corrected tip resistance greater than or equal to 160 tsf.
- Shear-wave-derived CRR: Seismic shear-wave velocities exceeding 660 ft/sec.

However, the bounding values are actually dependent on the fines content of the deposit. Figure 5.5 presents published curves for SPT-derived CRR, and shear-wave-derived CRR (Youd and Idriss 1997) showing the variation in the bounding value for different percent fines content. These curves were developed from empirical data and suggest that there are bounding values for CRR, above which the soils remain stable irrespective of the corresponding CSR. For example, Fig. 5.5 shows that, for any CSR value, an SPT-derived CRR of $N'_{60} \geq 30$ is stable, even for soils having very few fines (<5%).

In deriving these curves, researchers selected a magnitude 7.5 earthquake because it generally represents the upper bound for the empirical data used to develop these relationships. This magnitude also happens to represent the potential source of strong ground motions at Site 3A (BJC 2002c).

The CSR vs. CRR procedure was originally developed for evaluating site liquefaction potential where SPT data are used as a basis for CRR (Seed and Idris 1982). Therefore, the SPT N'_{60}-values measured in the soil borings at Site 3A were used in this quantitative evaluation. Over the past two decades, however, SCPT (and to a lesser extent shear-wave velocity measurements) have been increasingly used to assess liquefaction potential.

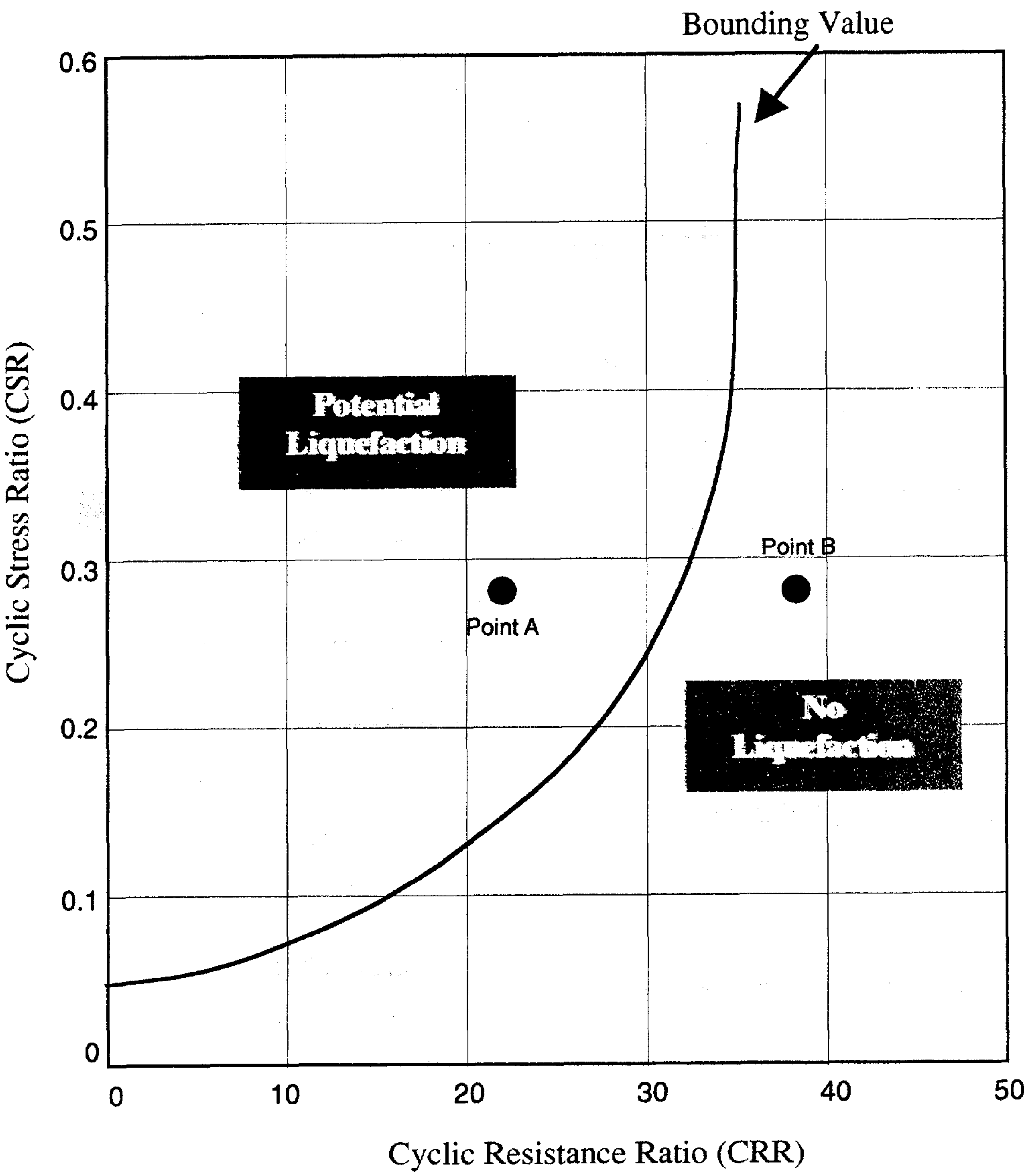

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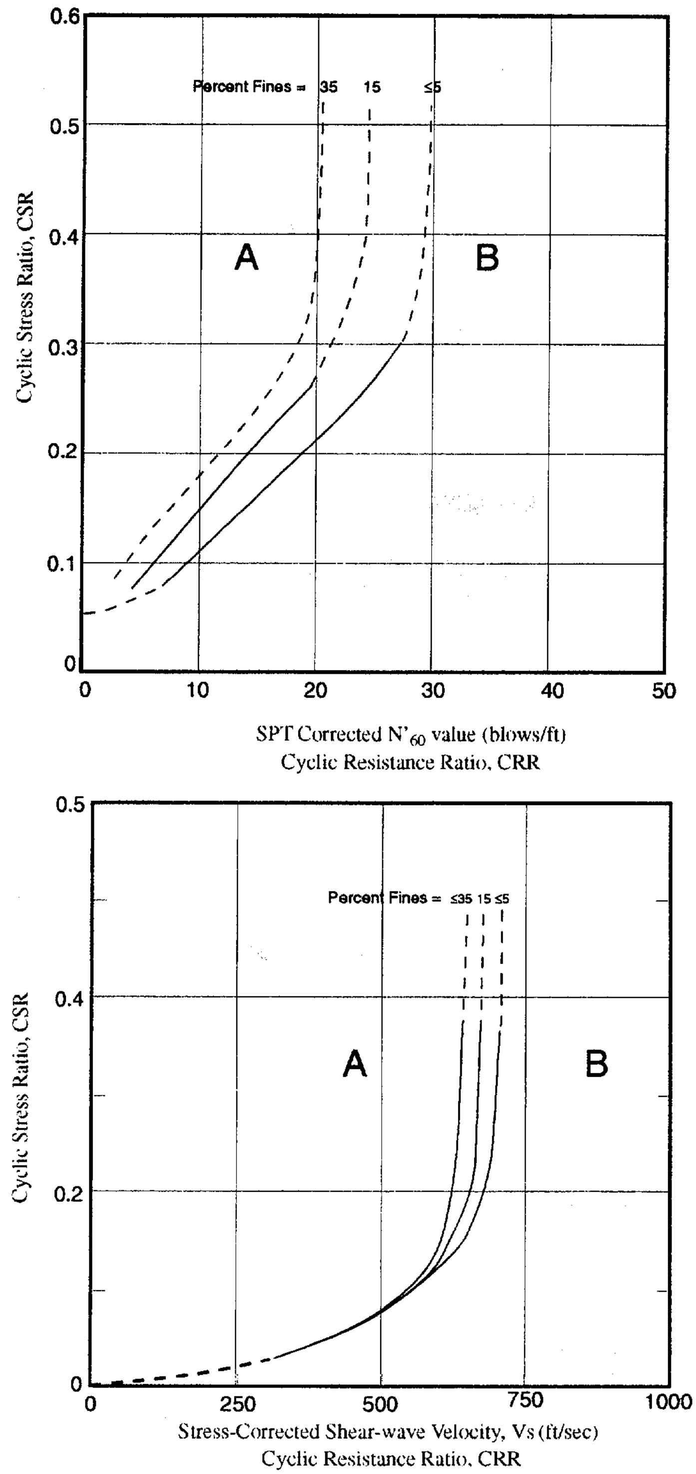
Fig. 5.4. Plot of CSR vs. CRR showing bounding value.

Figure No. 5.4
DATE 7/3/02



SPT...... Standard penetration test
SCPT....Seismic cone penetrometer test

Note: Curves shown are for an assumed magnitude 7.5 earthquake.

Fig. 5.5. Plots of CSR vs. CRR for SPT and shear-wave velocity.

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PADUCAH GASEOUS DIFUSION PLANT

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Figure No. 5.5
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The SCPT method has the advantage of providing continuous data with depth at relatively low cost. SCPT data can be used in two ways. Under the first approach, SCPT data are converted to equivalent N'_{60}-values using procedures such as described in (Robertson and Wride 1997). Following this conversion, the curve in Fig. 5.5 can then be used to assess liquefaction potential. Because most of the soils encountered at Site 3A are not clean sands, this approach of using SCPT data converted to N'_{60}-values was used more extensively in this evaluation.

A second approach in evaluating SCPT data uses the actual tip resistance values. This approach allows the calculation of CRR without the need to convert to equivalent N'_{60}-values and then applying SPT-derived criteria. This approach of using direct SCPT tip resistance values was, therefore, used as an additional check for the sands encountered in the SCPT tests at Site 3A.

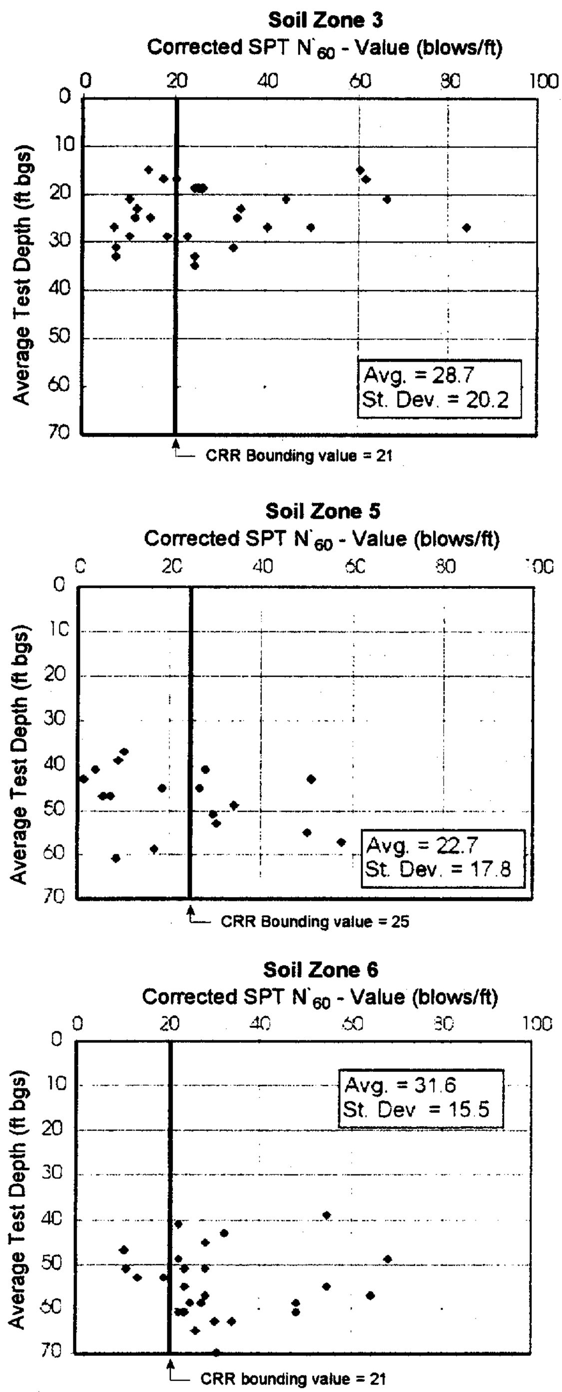
A final technique to assess liquefaction potential is the use of shear-wave velocities. The use of shear-wave velocity as an index of liquefaction resistance is appropriate because many of the same factors that affect liquefaction resistance also influence shear-wave velocity (Andrus and Stokoe 1997). The curves shown in Fig. 5.5 define the shear-wave-derived CRR for different percent fines content.

SPT results: Figure 5.6 presents the results of the bounding value evaluation using corrected N'_{60}-values for soil zones 3, 5, and 6. The vertical lines define the bounding values of SPT-derived CRR taken from Fig. 5.5 for percent fines most similar to those of the corresponding soil zone. A bounding value of 21 is shown for soil zones 3 and 6 because this is the bounding value for SPT-derived CRR for soils with >35% fines. A bounding value of 25 is shown for soil zone 5 because this is the bounding value for soil with ~20% fines. Results are summarized in the right-hand column of Table 5.3. Based on the SPT-derived CRR bounding values, 57% of the soils in zone 3 would be stable under the upper-bound earthquake load. Similarly 47% of the soils in zone 5 and 85% of the soils in zone 6 would be stable.

SCPT (converted) results: Figure 5.7 presents SCPT data converted to N'_{60}-values for soil zones 3, 5, and 6. Again, the vertical lines define the bounding values of SPT-derived CRR taken from Fig. 5.5 for the percent fines content most similar to that of the corresponding soil zone. Results are summarized in Table 5.3. Based on the SPT-derived CRR bounding values (for SCPT data converted to N'_{60}-values), 89% of the soils in zone 3 would be stable under the upper-bound earthquake load. Similarly, 41% of the soils in zone 5 and 76% of the soils in zone 6 would be stable.

SCPT (tip resistance) results: Figure 5.8 presents the results of the bounding value evaluation using direct SCPT tip resistance. Only those SCPT tip resistance values that are associated with sands or gravelly sands are shown, since fine-grained soils are not prone to liquefaction. Figure 3.10 presents a composite plot of all soil zones as a general check of liquefaction potential. Although all soil zones have been combined in this general check plot, the results are similar to those for SCPT data converted to N'_{60}-values, as shown in Fig. 5.9. Based on the SCPT-derived CRR bounding values for direct SCPT tip resistance, 74% of the sandy soil zones would be stable under the upper-bound earthquake load.

Shear-wave velocity results: Figure 5.9 presents the results of the bounding value evaluation using shear-wave velocities for soil types 3, 5, and 6. The vertical lines define the boundary values of shear-wave-derived CRR taken from Fig. 5.5 for the percent fines content most similar to that of the corresponding soil zone. Results are summarized in Table 5.3. Based on the shear-wave-derived CRR bounding values, 100% of the soils in zones 3 and 5, and 92% of the soils in zone 6 would be stable under the upper-bound earthquake load. With very few exceptions, the measured shear-wave velocities at Site 3A all lie to the right of the vertical line that defines the boundary value CRR, suggesting that liquefaction is not likely.



Note: Areas to the right of the CRR bounding value line are stable.
Areas to the left could experience liquefaction.

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Fig. 5.6. CRR bounding value vs. observed values for SPT data.

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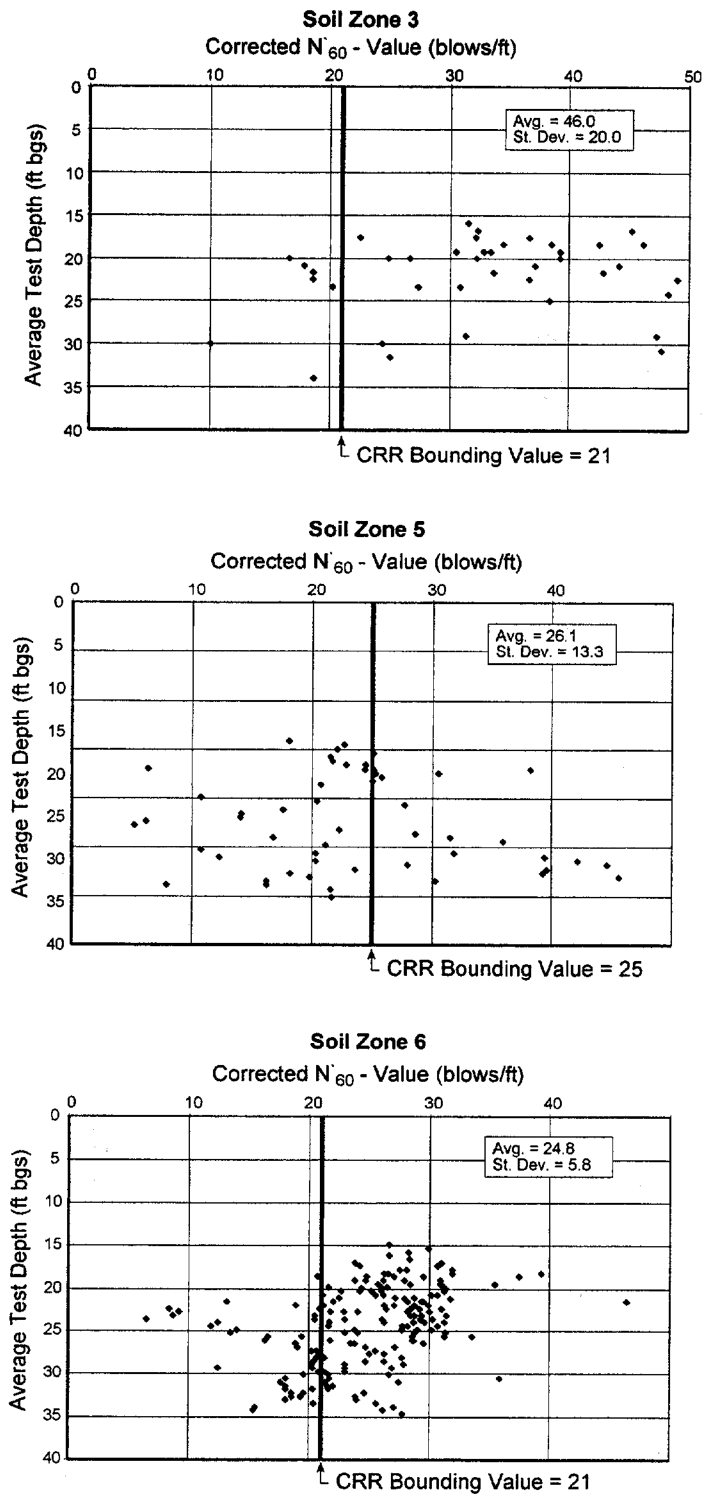
Table 5.3. Quantitative evaluation of liquefaction potential

Soil zone	Average percent fines	USCS classification	Average depth (ft bgs)	Average CRR	Median CRR	Number of tests	Percentage of tests falling within stable range: Peak ground acceleration (PGA) 0.1g	0.2g	0.3g	0.4g	0.5g	Bounding value
SPT N'_{60} data (CRR based on N'_{60})												
3	41	SP-SC	24	28.7	24.4	28	100	89	68	61	61	57
5	20	SP-SC	48	22.7	18.3	17	88	59	47	47	47	47
6	43	SM	55	31.6	27.7	26	100	100	88	85	85	85
SCPT data converted to N'_{60}-values (CRR based on N'_{60}-values)												
3	41	SP-SC	23	46.0	42.7	64	100	100	98	91	89	89
5	20	SP-SC	33	26.1	22.8	56	100	89	73	48	46	41
6	43	SM	49	24.8	25.7	176	100	98	93	84	83	76
Shear-wave velocity data (CRR based on shear-wave velocity)												
3	41	SP-SC	23	1249.9	1204.3	19	100	100	100	100	100	100
5	20	SP-SC	49	1068.0	1006.1	19	100	100	100	100	100	100
6	43	SM	51	1028.0	1002.9	48	100	100	100	100	100	92

CRR = cyclic resistance ratio
N'_{60} = corrected SPT blow counts
SC = clayey sand
SCPT = Seismic Cone Penetrometer Test
SM = silty sand
SP = poorly-graded sand
SPT = Standard Penetration Test
USCS = Unified Soil Classification System (ASTM D1586)



Note: Areas to the right of the CRR line are stable. Areas to the left could experience liquefaction.

Fig. 5.7. CRR bounding value vs. observed values for converted SCPT data.

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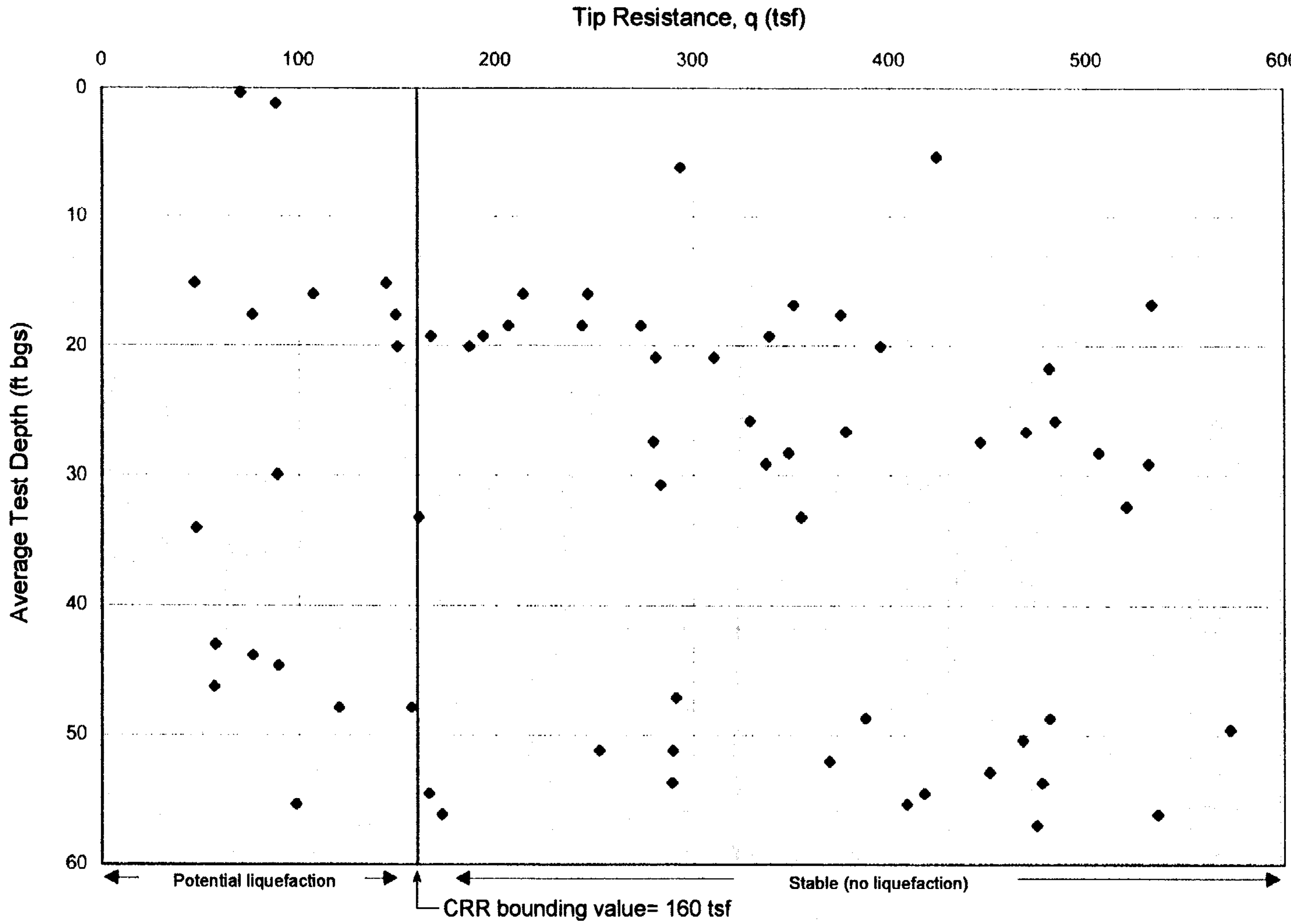

tsf = tons per square foot
CRR = cyclic resistance ratio

Fig. 5.8. CRR bounding value vs. observed values for SCPT tip resistance data.

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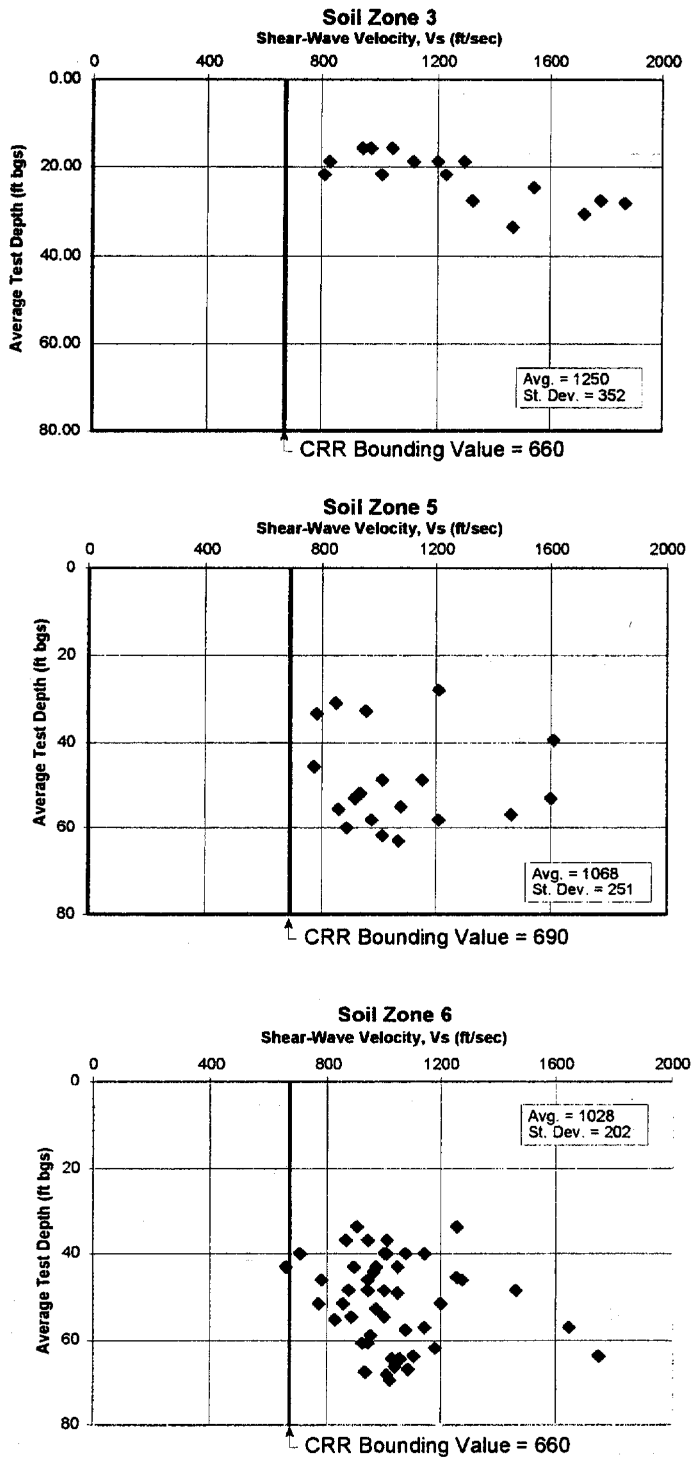

Note: Areas to the right of the CRR line are stable. Areas to the left could experience liquefaction.

Fig. 5.9. CRR bound value vs. observed value for shear-wave velocity data.

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The curves in Fig. 5.5 show that as the percentage of fine-grained materials increases, the curve separating liquefiable vs. non-liquefiable areas shifts to the left, and the bounding CRR required to prevent liquefaction becomes lower. While these curves are most applicable for sands (fines content less than 5%), and silty sands (fines content 15 to 35%), their use to evaluate the liquefaction potential of soils with even higher percentages of fines (greater than 35%) is more conservative. For soils in zone 5, the average percent fines (as described in Chap. 3) is approximately 20%, so that the evaluation is valid. For soils in zones 3 and 6, the average percent fines is approximately 41 to 43%, so that the evaluation is more conservative. The potential for liquefaction in these soil zones may, therefore, be overestimated.

5.5.3 Step 3: Quantitative Evaluation of Liquefaction Potential

The Step 2 screening done in Sect. 5.5.2 evaluated CRR values using an upper-bound scenario where liquefaction would not be expected to occur even during earthquakes of high magnitude and strong ground motions. Where lower ground accelerations occur, CRR values are encountered that fall to the left of the bounding values. In these cases, the assumed earthquake loading conditions, groundwater levels, soil type, and depth need to be quantitatively assessed to better define the liquefaction potential. Step 3 screening, therefore, evaluates the liquefaction potential for lower ground motions.

The local groundwater levels and soil density reported in Chap. 3 for Site 3A were used to develop an estimate of the CSR that might be expected during lower ground motions. Using equation 5-1, CSR values were calculated for each of the three granular soil zones (3, 5, and 6) and for PGAs ranging from 0.1 g to 0.5 g. The highest value is based on the approximate maximum PGA presented in Chap. 7. The lower values represent ground motions that are more realistic, that is, could occur more frequently at Site 3A.

The CRR for each soil zone was then developed using the relationships shown in Fig. 5.5, and these CRRs were compared to the results of the SPT, SCPT, and seismic shear-wave velocity surveys presented in Chap. 3. The soil types, their percent fines, average depth, average and median CRR, and liquefaction potential under the range of ground motions (PGAs) listed above are presented in Table 5.3. The following summarizes the results of this evaluation.

SPT Analysis: Table 5.3 shows the results using SPT data. At low PGA values, between 80 and 100% of the soils would remain stable in all soil zones. At PGAs approaching 0.5 g, there is a greater potential for liquefaction to occur. Based the SPT data, between 47 and 85% of the soils would remain stable at a PGA of 0.5 g.

SCPT Analysis: Table 5.3 shows the resulting using SCPT data converted to N'_{60}-values. At low PGA values, 100% of the soils remain stable. At PGAs approaching 0.5 g, there is a greater potential for liquefaction to occur. Based on the SCPT data, between 46 and 89% of the soils would remain stable at a PGA of 0.5 g.

Shear-Wave Velocity Analysis: Table 5.3 shows the results for shear-wave velocity data. Based on the shear-wave velocity data, there is very little potential for liquefaction to occur under the full range of PGAs. Even at PGAs approaching 0.5 g, 100% of the soils would remain stable.

5.5.4 Summary of the Liquefaction Potential at Site 3A

Many of the soils present at Site 3A are clays and silts that by their very composition are not prone to liquefaction. A screening evaluation of the seismic and geotechnical properties of the soils at Site 3A concluded that soils in zones 1, 2, and 4 are not prone to liquefaction, nor are they susceptible to severe strength loss similar to liquefaction. These soil zones would remain stable under any earthquake load.

A quantitative evaluation was performed for the remaining granular soil zones (3, 5, and 6) using bounding criteria, above which liquefaction would not be expected to occur even during earthquakes of high magnitude (approximately 7.5 g) and strong ground motions. Based on this upper-bound scenario, it is concluded that the soils would remain relatively stable; however, comparison of different data produce different results. Shear-wave velocity data suggest all three soil zones are stable. SPT and SCPT data suggest that some limited liquefaction could occur, particularly in soil zone 5. This analysis is conservative and may overestimate actual liquefaction potential.

A subsequent quantitative evaluation was performed for soil zones 3, 5, and 6 using a range of earthquake loading conditions for PGAs from 0.1 g to 0.5 g. Based on this range of ground motions, it is concluded that the soils would remain stable and would not be expected to liquefy under low to moderate levels of ground motion. However, some limited liquefaction within some of the sands and deformation within some of the silts and clays could occur at PGAs approaching 0.5 g.

5.6 SUMMARY OF THE LIQUEFACTION EVALUATION

There is no definitive evidence of paleoliquefaction at PGDP. Paleoliquefaction is defined here as seismically-induced liquefaction features associated with prehistoric Holocene or late Pleistocene earthquakes. Field investigations conducted along portions of Bayou and Little Bayou Creeks found no definitive evidence of paleoliquefaction on DOE property.

Field investigations conducted as part of the Seismic Investigation found no obvious liquefaction features along the Ohio River in the vicinity of the PGDP. The riverbank afforded adequate exposure of the sediments such that if large liquefaction features were present they would have been obvious. Smaller-scale paleoliquefaction features may have been present but were not observed because of their relatively small size or the typical veneer of river deposits and vegetative cover.

The literature does report some liquefaction features within 15 miles of PGDP. The closest are located along the banks of the Ohio River, about eight miles to the northeast. These features are in the general vicinity of Fort Massac, Illinois, a location where liquefaction was reported during the February 7, 1812, New Madrid earthquake. These features were small and relatively unweathered, suggesting that they were probably outlying liquefaction features resulting from the 1811 and 1812 New Madrid earthquakes. Small liquefaction features are also reported in the literature along the Post Creek Cutoff, about 12 miles northwest of the PGDP.

The absence of large liquefaction or paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. In this context "local strong ground motion" is taken to mean strong ground motion resulting from a local earthquake. The small liquefaction features that have been observed are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.

Many of the soils present at Site 3A are clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet criteria that distinguish those fine-grained soils that could experience large-scale strength loss, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. However, based on calculations presented in this report, it is concluded that some limited liquefaction within some of the sands and deformation within some of the silts and clays could occur at PGAs approaching 0.5 g.

6. EVALUATION OF FAULTING

As discussed in Chap. 1, the Project Core Team has developed a list of seven questions that, when answered, would address seismic issues related to the siting of a potential CERCLA waste disposal facility at PGDP. This chapter reports on studies designed to address Questions 4 and 5 posed by the Project Core Team. These two questions follow:

- Question 4: Is there evidence of Holocene displacement of faults at PGDP?
- Question 5: Are there faults underlying the potential disposal facility site?

Table 6.1 repeats the questions and provides the answers based on these investigations. Data, information, and details used to develop the responses presented in Table 6.1 are included in the body of this chapter. The Fault Study performed as part of the Seismic Investigation program for Site 3A included both a regional and a site-specific component. Results and interpretations made in each of these studies are described in the following sections. To provide the proper content for these discussions, a brief background on faulting precedes them.

Table 6.1. Summary of answers to Questions 4 and 5 posed by the Project Core Team

Question	General answer
4. Is there evidence of Holocene displacement of faults at PGDP?	This study did not find Holocene displacement of faults at Site 3A. Several faults identified in seismic reflection data at 3A have been confirmed to extend through the Porters Creek Clay and into the materials underlying the surficial loess deposits. Three of these faults are interpreted to extend to within approximately 20 ft of the ground surface. One DPT borehole encountered three fault planes at depths between 22 ft and 28 ft. No faults were observed in the overlying loess. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age with ^{14}C dates ranging from 13,500 to 15,600 years BP. At the Barnes Creek site located 11 miles northeast of PGDP this study found Holocene age displacement of faults in deposits with ^{14}C dates ranging from 5,000 to 7,000 years BP.
5. Are there faults underlying the potential disposal facility site?	The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults beneath Site 3A, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks. Several of the faults extend through the Porters Creek Clay and into the materials underlying the surficial loess. Three of these faults extend to within 20 ft of the ground surface.

BP = years before present, where "present" is defined as 1950 A.D.
DPT = direct push technology
PGDP = Paducah Gaseous Diffusion Plant

6.1 BACKGROUND ON FAULTING

Faulting is a process by which tectonic forces in the earth's crust cause movement in rock, typically as a result of an earthquake. This faulting results in a planar surface, or fault, across which there has been observable displacement of the rock (i.e., where one side of the rock mass has moved relative to the other side). Depending on the relative direction of displacement on either side of the fault, its movement is described as normal, reverse or strike-slip.

In a normal fault, the side above the fault slides downward relative to the side below the fault. A reverse fault is just the opposite; it forms when the side above the fault slides upward. A strike-slip fault is one where the rock masses slide horizontally with respect to one another. These faults are fairly steep, and can be nearly vertical.

In areas under tension, the rocks are being pulled apart by tectonic forces. In these areas, called "rift zones", groups of normal faults can form, producing what is known as horst and graben topography. A horst is a relatively high-standing area formed by the movement of normal faults that dip away from each other. The result can be seen as an apparent up-thrown rock mass or an arch-shaped rock surface. A graben, on the other hand, is a relatively low-standing area formed by the movement of normal faults that dip toward one another. The result can be seen as a down-dropped block or bowl-shaped rock surface.

Old faults that formed in one type of stress field are commonly re-activated or serve as the location of renewed movement under new stress orientations. For example, while normal faults are usually formed in areas under tension, they can experience strike-slip motion if the stress field changes.

Faults also can occur as extensions of the rock faults upward into the soil profile. Faults in soil can also be normal, reverse, or strike-slip, and can also cause horst and graben features. These are the types of faults investigated in this Seismic Investigation.

6.2 REGIONAL FAULT STUDY

During a scoping meeting held on June 6 and 7, 2001, DOE, EPA, and the Commonwealth of Kentucky agreed that the purpose of the regional Fault Study is to collect data that will support designing a potential on-site CERCLA waste disposal facility. In a follow-up workshop held on July 11 and 12, 2001, the Project Core Team, in consultation with subject-matter experts including representatives from the Kentucky Geological Survey, ISGS, Mid-America Earthquake Center, and the University of Memphis, selected a site in Massac County, Illinois, for the regional Fault Study. The site selected is located at and near Barnes Creek, approximately 11 miles northeast of PGDP (Fig. 6.1).

The Barnes Creek site was selected because it has been extensively investigated previously by the ISGS, so that a great deal of information was already known about the faulting there (Nelson et al. 1997). In addition, faulting is readily visible in the deposits exposed on the banks of the Barnes Creek channel. This provides geologists the opportunity to determine whether faulting extends into the geologically young sediments over a broad exposed stretch of the creek. Finally, as discussed in Sect. 3.2, the Barnes Creek fault zone, if extended below the Mississippi Embayment, would be the most likely fault system to pass through or near PGDP. Therefore, evaluation of faulting at Barnes Creek may provide valuable data for supporting design of any potential CERCLA waste disposal facility at Site 3A.

A group of southwest-trending fault zones termed the Fluorspar Area fault complex extend through the area of the Barnes Creek site, as shown on Fig. 3.2. The Fluorspar Area fault complex is composed mostly of high-angle normal faults that outline horsts and grabens. High-angle reverse and oblique-slip faults also are present in the bedrock and reflect multiple episodes of movement. Vertical dip-slip offsets in Paleozoic rocks are as great as 2250 ft; the horizontal strike-slip component is undetermined. The locations and trends of the geologic structures in the Barnes Creek channel suggest that they may represent the southwestern extensions of these bedrock faults (Fig. 3.3). A seismic reflection profile (Sexton et al. 1996) shows that faults exposed in the streambed are upward extensions of high-angle normal and reverse faults in Paleozoic bedrock.

At the time when these normal and reverse faults were originally formed, the bedrock was in tension, pulling itself apart; but today the bedrock is in compression, pushing itself together. Therefore, the stresses that caused the faults to develop no longer exist. However, some of the faults have likely been reactivated as strike-slip faults under the current compressional stress regime.

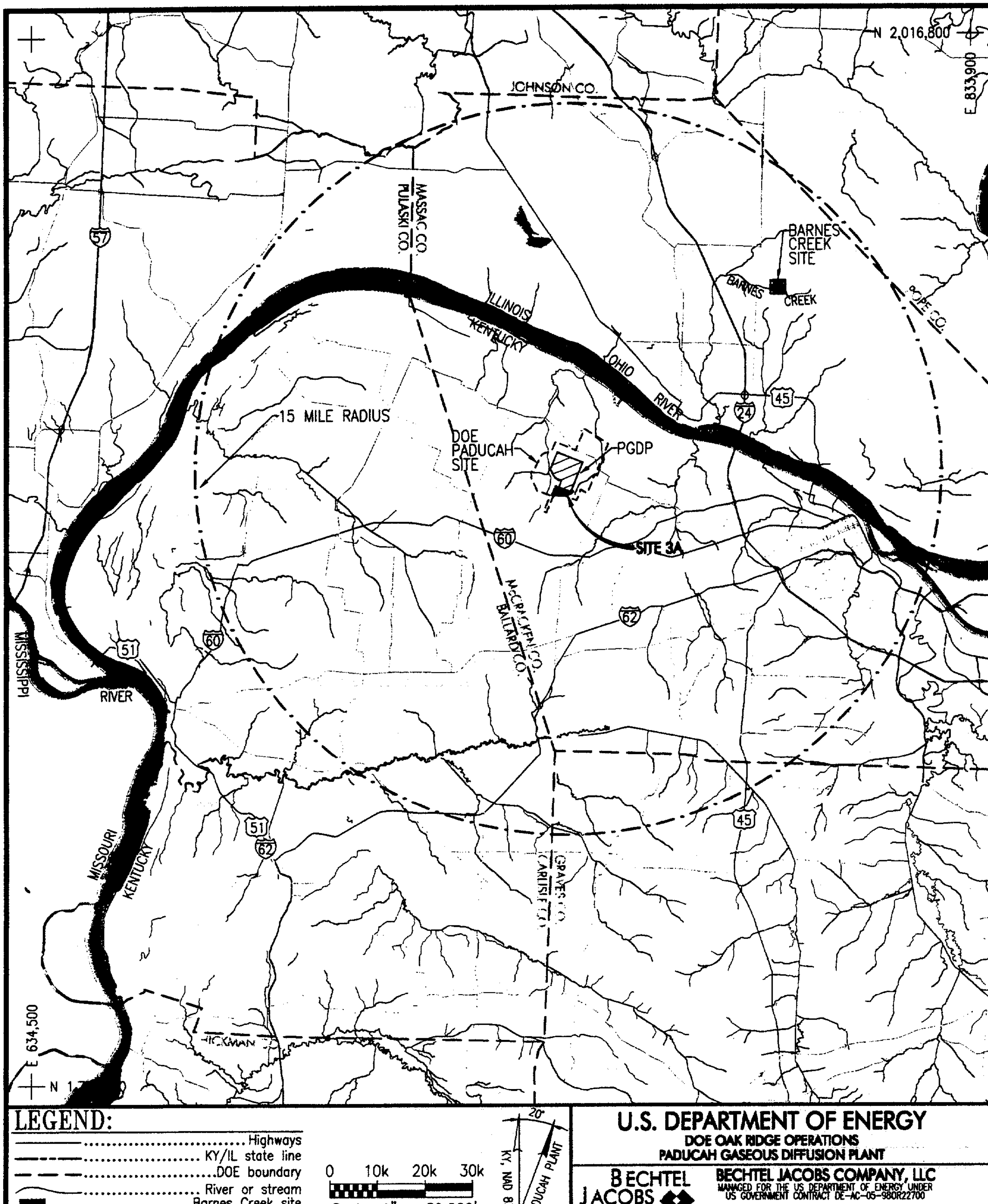

Fig. 6.1. Location of PGDP and Barnes Creek site.

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The Cretaceous McNairy Formation overlies the bedrock. Unlike at Site 3A, where the McNairy Formation is covered by approximately 155 ft of sediments, at Barnes Creek much of these sediments have been eroded away and the McNairy is exposed near the base of the creek (Fig. 6.2). The McNairy, is covered by about 10 to 20 ft of relatively young sediments along the creek banks.

Barnes Creek drains a wide floodplain. Prior to the 1920s, the creek meandered across the low-lying area and cut several scarps or "cut banks" in the adjacent terraces. Figure 6.3 shows some of the probable old stream channels. A man-made channel was excavated in the 1920s or 1930s that permanently realigned the creek to its present location. Since that time, the increased gradient of the man-made channel has resulted in downward cutting of the streambed.

The regional Fault Study included two principle activities: (1) detailed mapping of a 2600-ft portion along Barnes Creek and (2) GPR and DPT investigations of a suspected nearby graben, referred to as the "terrace graben" (Fig. 6.4). During this investigation more than 300 digital photographs were collected, documenting the stratigraphy, faulting, and collection of organic samples for radiocarbon (^{14}C) age dating. These have been organized into a CD-based Web page that can be viewed in an interactive mode. Figure 6.5 shows the home page of this CD-based support tool. By loading the CD and clicking at the various locations highlighted on the home page, the reader can view all of the photographs taken in their proper context. A series of panoramic views at each of the "stops" mapped during this investigation can also be viewed, panned, and zoomed for closer inspection of the stratigraphy and faulting. A photograph of each in-place ^{14}C sample is also included to document this aspect of the study.

The approach used in the Barnes Creek bank study consisted of the following four components:

- identify the key geologic units and their relationship (local stratigraphy),
- identify and characterize faults and other geologic structures,
- identify each geologic unit's relationship with the observed faults, and
- date the geologic units to establish ages of past movements.

These components were accomplished by systematic examination of both banks of Barnes Creek in the study area. A local survey baseline was set up to aid in identification of stationing for the features studied (Fig. 6.4). In areas of interest, scraping of surface materials, roots, and other vegetation was completed to provide continuous exposure. Work was completed by SAIC geologists and, for two days, with the help of scientists from the ISGS. A scientist from the University of Memphis aided in the collection of charcoal samples for radiocarbon age dating.

In the terrace graben area, approximately 2700 ft of GPR survey data were collected. Ten DPT boreholes were then cored to better define the limits of the potential terrace graben and its boundary faults. In addition, a total of five samples were collected for radiocarbon age dating.

6.2.1 Stratigraphy at Barnes Creek

Figure 6.3 presents the regional stratigraphic column as presented by Nelson et al. 1997. The figure illustrates the ages and the typical thickness of the units in the Massac County, Illinois, study area. The younger silt units (Peoria, Roxana, and Loveland) are loess deposits formed by wind-blown dust during inter-glacial periods. These loess deposits are easily recognizable units and serve to aid in stratigraphic interpretation. Although preserved on the higher terraces in the area surrounding Barnes Creek, the loess deposits have not been identified in the creek itself.

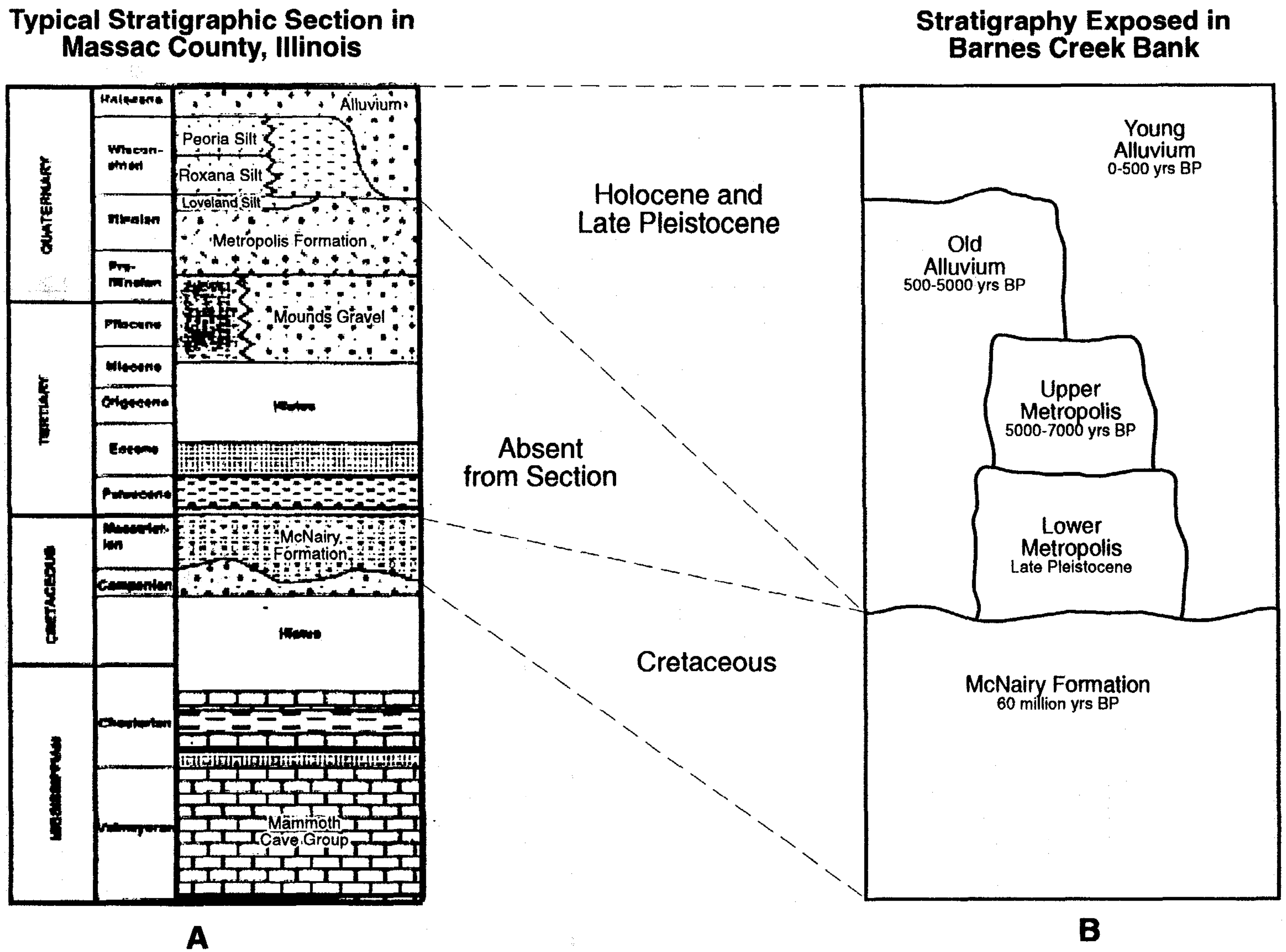

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Fig. 6.2. Regional and local stratigraphy.

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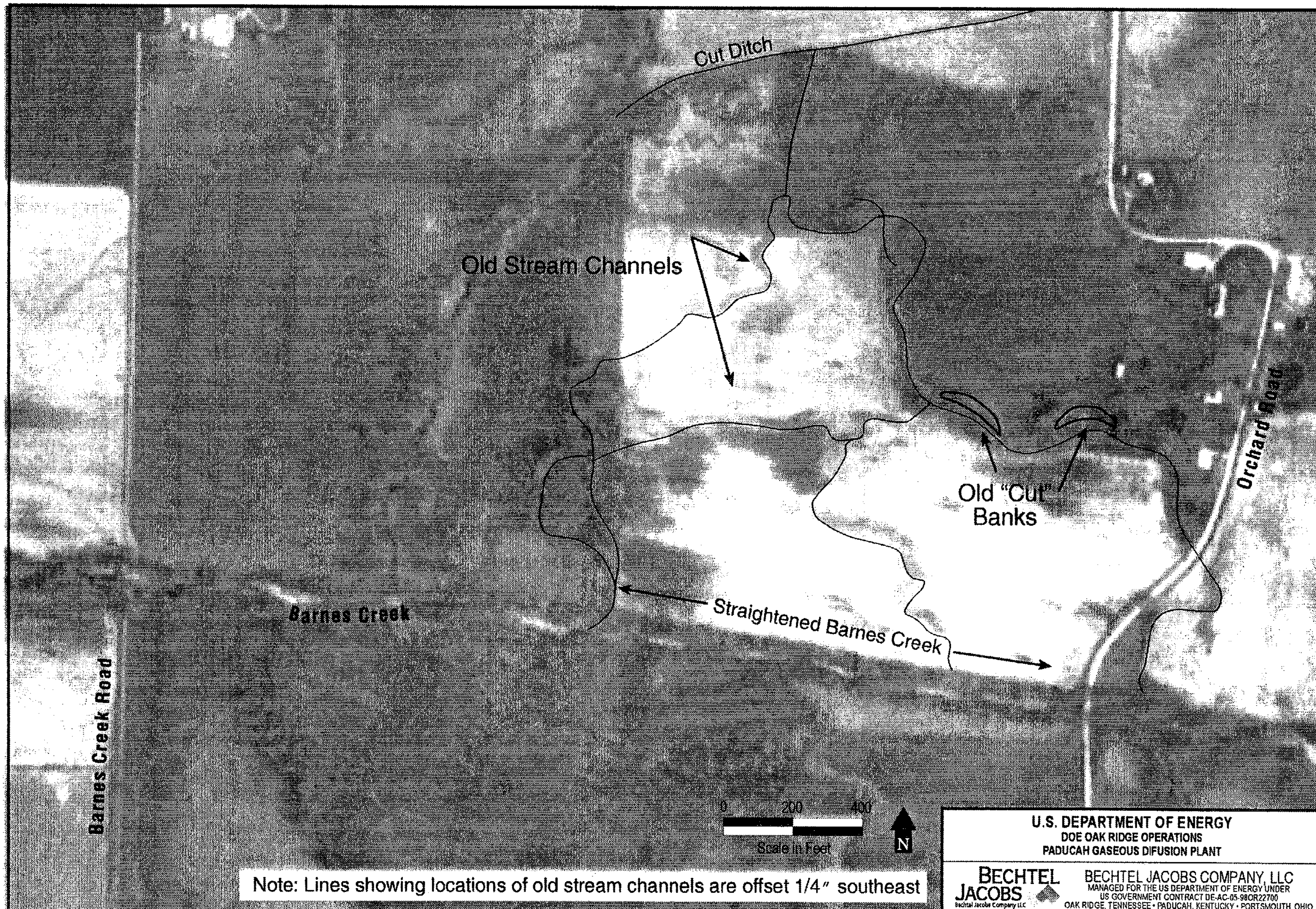

Fig. 6.3. Overview of Barnes Creek site.

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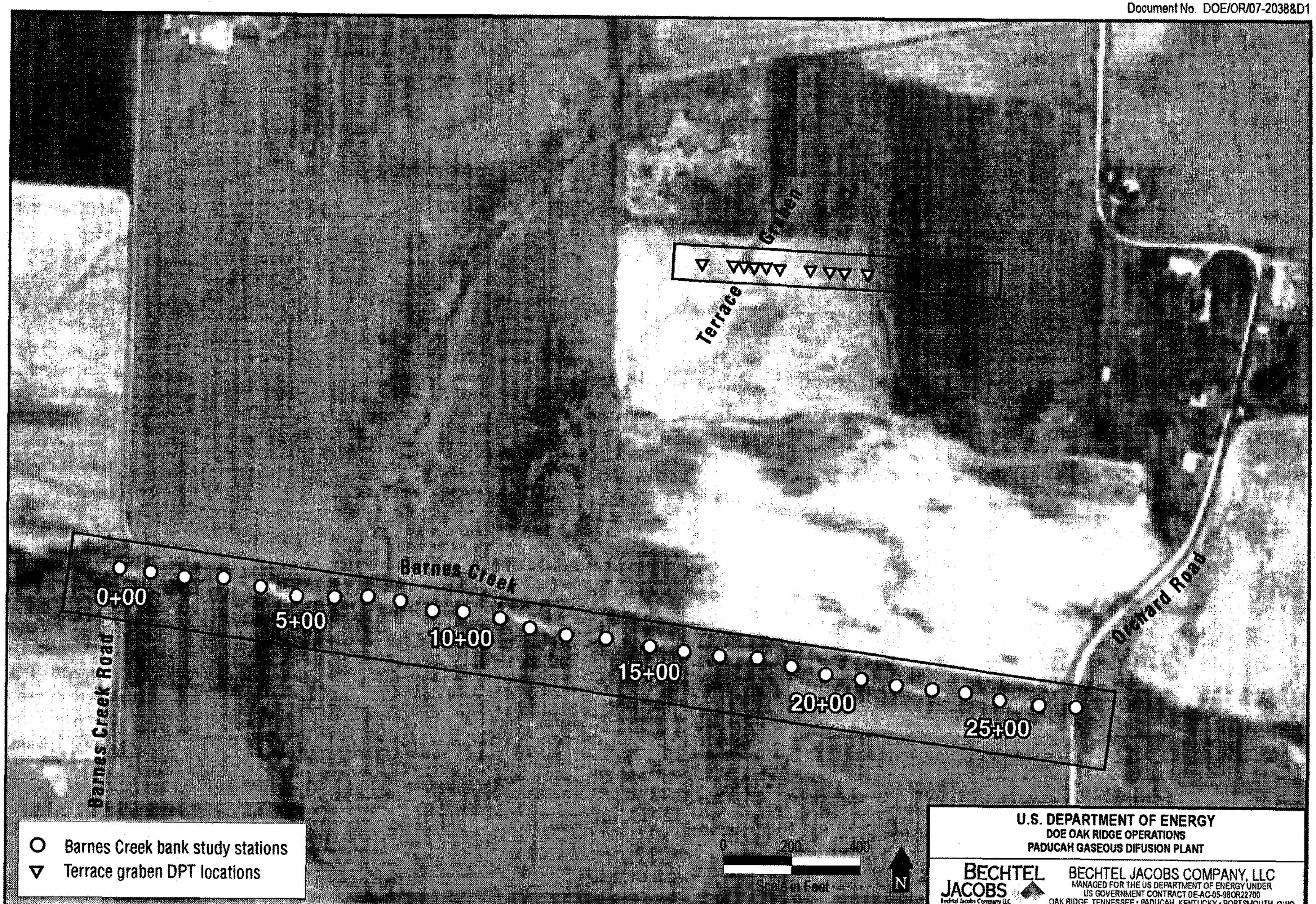

Fig. 6.4. Locations of bank study and terrace graben area.

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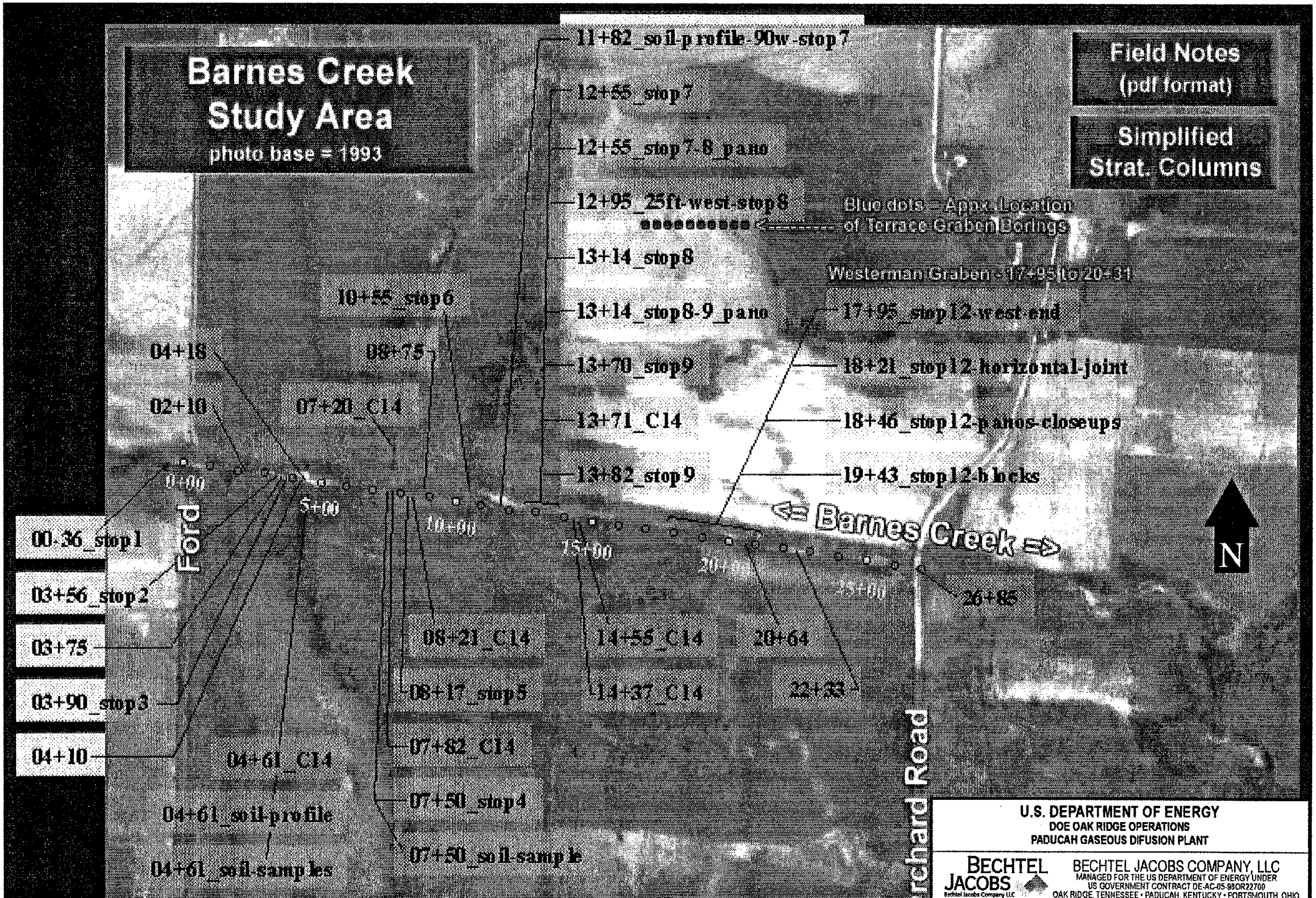

Fig. 6.5. "Home Page" of interactive CD.

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Within Barnes Creek itself, four stratigraphic units were identified as follows.

- young alluvium
- old alluvium
- the Metropolis Formation
- the McNairy Formation

For the purposes of this study, the Metropolis Formation was also subdivided into an "upper" and "lower" unit. Each of these units is described below and shown on Fig. 6.3. Figure 6.3 also gives general age estimates based on radiocarbon age dating performed during this investigation. A photograph of a section in Barnes Creek where all of these units are present is shown as Fig. 6.6. Each of the units is described below.

Young Alluvium: This material caps or is exposed in virtually all of the creekbank to a thickness generally exceeding 2 to 6 ft. It is composed of soft, unconsolidated light brown sandy silt, silty sand, and sand with minor clay or small gravel in some areas. The internal structure varies from massive to weakly bedded. The upper portion of the unit contains abundant modern roots and rootlets although no soil profile formation has occurred. Three ^{14}C dates are available from this material; they indicate that this unit has been deposited within the past 500 years. As represented in Fig. 6.3b, this unit is present along the entire stretch of Barnes Creek. Because of local erosion that took place prior to its deposition, the young alluvium can lie directly over any of the older units.

Old Alluvium: In several areas within Barnes Creek, an older, deeply weathered alluvium has been observed. This material is typically light gray and contains numerous small iron nodules or stains yielding a spotted appearance (Fig. 6.6). Some larger scale (6 to 18 inches) darker gray mottling is observed in some areas. The unit generally appears as a massive fine sand or silty fine sand. The thickest exposures of this unit are approximately 6 ft. Obvious bedding planes or internal structures such as cross bedding or cut-and-fill have not been observed. Indistinct bedding on a scale of 6 inches to 1 ft has been observed as small changes in the relative percentages of sand and silt and minor clay. As represented in Fig. 6.3b, this unit is not present along the entire stretch of Barnes Creek. When present, it is always covered by the young alluvium. Because of local erosion that took place prior to its deposition, the old alluvium can lie directly on the upper Metropolis, lower Metropolis, or McNairy sediments.

It is possible that this unit has been thoroughly disturbed (e.g., by tree roots or animal burrows) such that any originally internal structure has been obliterated. In some outcrops, thin, minor gravel appears in the uppermost part of the section. There are almost always several inches of loose gravel at the base of this formation. A total of two organic samples were collected from within this unit; the dates indicate that this unit was deposited within the past 5000 years. Because the young alluvium caps this unit and it is heavily weathered, it is thought to be at least 500 years old.

Metropolis Formation: Nelson et al. 1997 mapped the gravel-rich unit in Barnes Creek as the Metropolis Formation, a fluvial terrace deposit of Pleistocene age. As described in Nelson, the typical Metropolis Formation is "principally silt and sand, with lesser amounts of clay and gravel. The unit is largely unsorted to poorly sorted and massive to indistinctly bedded. Strong mottling in yellowish gray, brown, orange and red is characteristic. The Sangamon Geosol is at the top and marks the upper boundary. Thick, multiple weathering profiles are developed below the Sangamon at many outcrops." Also, the identification of the Metropolis Formation is more certain in areas where is it is clearly overlain by the interglacial loess (wind-blown silt and sand) deposits of the Peoria, Roxana, and Loveland Silts.

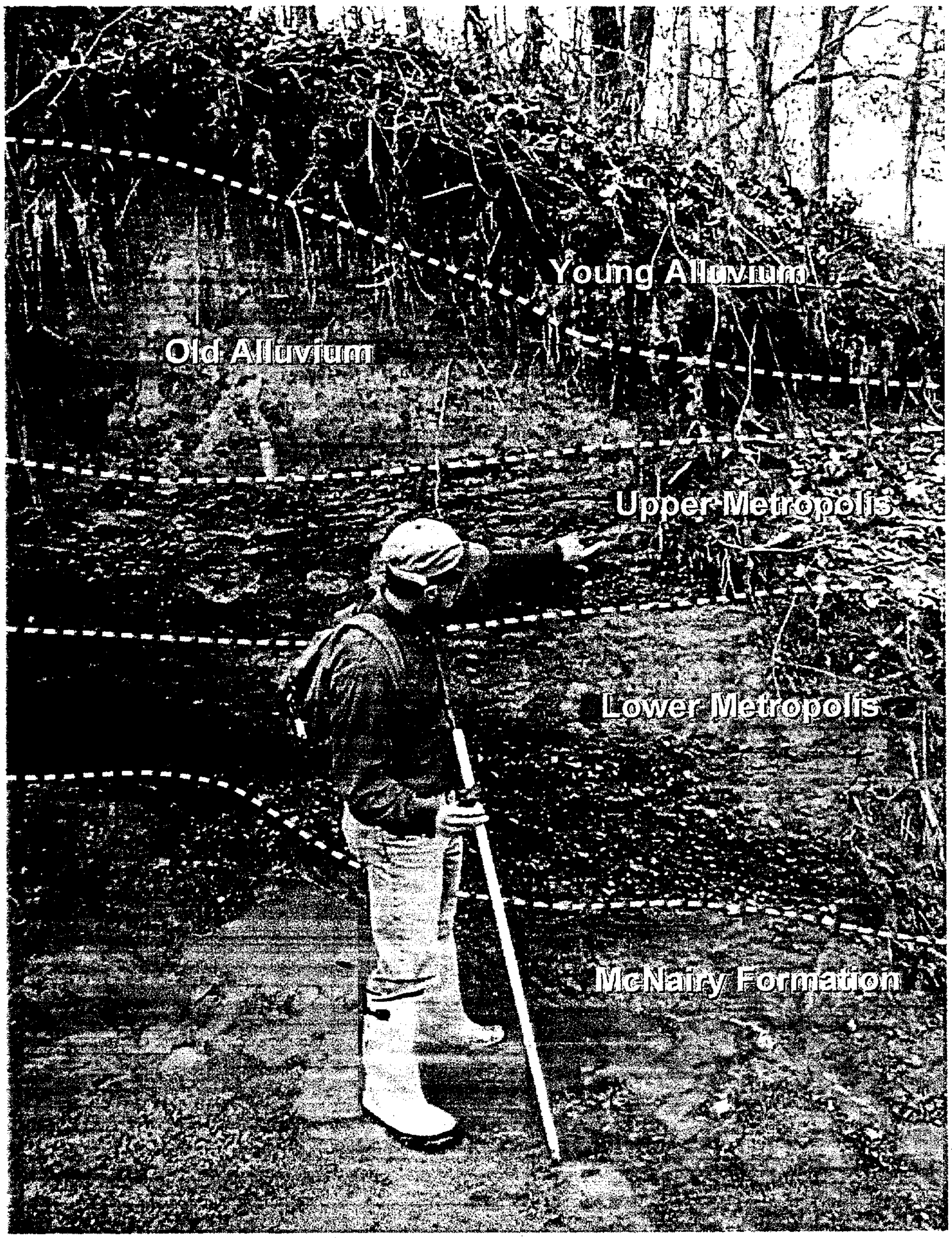

Fig. 6.6. Stratigraphic section at Barnes Creek.

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The typical grain size of the Metropolis Formation and the existence of multiple weathering profiles suggest deposition over a long period of time by slowly meandering sluggish streams typical of fluvial terrace deposition. Work in Barnes Creek during this study suggests that the unit originally mapped as Metropolis in Barnes Creek by Nelson in 1999 may not actually be the Metropolis Formation. This conclusion is based on several observations, as follows: First, exposures in Barnes Creek are typically 50 to 80% gravel units, with minor interbeds of sandy or clayey material. This is the inverse of the more typical Metropolis grain-size distribution. Second, no soil profiles were observed within the Metropolis exposures in Barnes Creek, in contrast to the numerous soil profiles observed elsewhere. Third, the finer-grained subunits in Barnes Creek are not extensively mottled or burrowed, although some mottling and burrowing was observed. While not conclusive, these field observations suggest that within Barnes Creek, the gravel-rich deposits originally mapped as Metropolis Formation are younger, re-worked, and re-deposited materials derived from Metropolis outcrops upslope. As discussed below, radiocarbon dates support this interpretation. In his 1999 study, Nelson noted that the age of this unit was uncertain.

For the purposes of this study, however, this unit will be referred to as the Metropolis Formation. This will correlate with previous researcher's nomenclature and will be consistent with the dating information collected during this investigation. Within Barnes Creek, the Metropolis has been further subdivided into the upper Metropolis and the lower Metropolis.

The upper Metropolis consists of interbedded gravel, sand, and silt layers and lenses indicative of small scour and cut-and-fill structures (Fig. 6.6). Individual beds are generally less than 6 inches thick and less than 10 ft wide. Total thickness of the upper Metropolis is rarely more than 1 to 2 ft. As represented in Fig. 6.3b, this unit is not present along the entire stretch of Barnes Creek; however, when present it always overlies the lower Metropolis. A total of five organic samples were collected from within this unit. The dates indicate that it was deposited between 5000 to 7000 years ago.

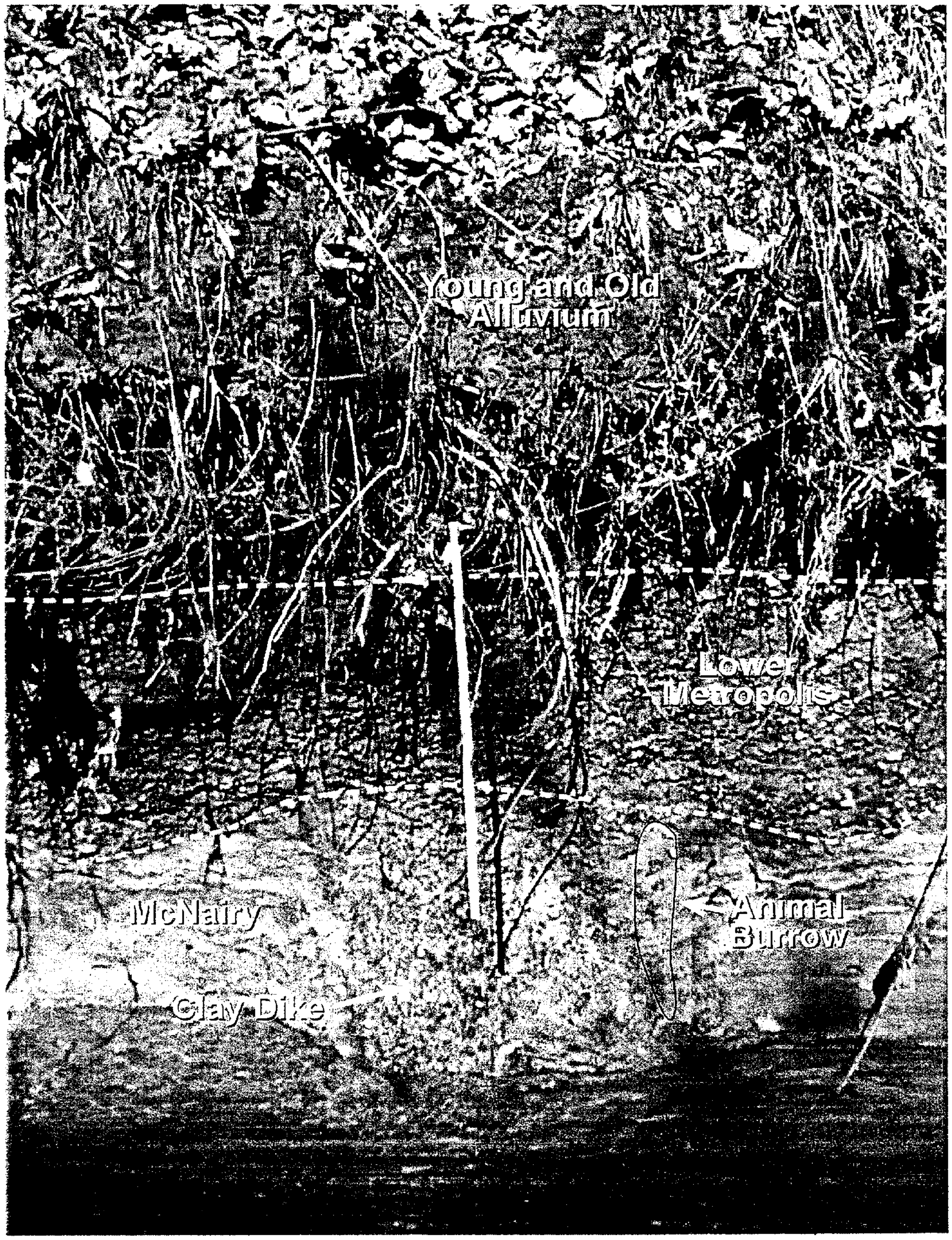
The lower Metropolis is predominately massive to weakly bedded gravel deposits, with flattened and rounded chert and quartz clasts ranging from 1 to 2 inches in diameter (Fig. 6.6). The gravel appears generally matrix supported, with a matrix of sand and silt. Except where thickened in the grabens, the lower Metropolis is generally less than 3–4 ft thick. No organic samples were found within the lower Metropolis. However, because the upper Metropolis overlies this unit, it can be concluded that it is at least 7000 years old. Radiocarbon dates from the terrace graben area (discussed in subsequent sections) suggest that it probably dates to the late Pleistocene.

McNairy Formation: The oldest exposed materials in Barnes Creek belong to the Cretaceous-aged McNairy Formation, which is many tens of millions of years old. This formation ranges from finely laminated to massively bedded. It is composed of beds of clay, silt, and fine sand. Many of the units are highly micaceous (Nelson et. al. 1996). The McNairy is present at or near the low-water line along much of Barnes Creek (Fig. 6.6). In some areas, it reaches as much as 4 to 5 ft up the bank (Fig. 6.7). Except in areas where it has been faulted, the bedding planes of the McNairy Formation are near horizontal, although some gentle folding increases dips to 5 to 10 degrees in some areas.

6.2.2 Geologic Structures at Barnes Creek

Geologic structures in Barnes Creek include individual joints, faults, clay dikes, and paired faults forming down-dropped blocks known as grabens. Each of these types of structures is described below.

Joints, Faults, and Clay Dikes: A number of individual joints, faults, and clay dikes have been identified in the study area. As used in this report, the term "joint" refers to a distinct planar break across which there has been no relative movement. The term "fault" refers primarily to a distinct plane along



Example of clay dike in the McNairy. The lower Metropolis lies over this feature and shows no deformation, indicating that motion took place prior to its deposition. The curved gray shape to the right of the clay dike is an infilled animal burrow.

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Fig. 6.7. Example of clay dike.

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which relative movement has clearly taken place. The term "clay dike" is applied to features that appear as well defined clay-filled or clay- and gravel-filled planar features usually several or more inches thick, but usually with no discernable displacement of the geologic units on either side. They are thought to represent cracks or "pull-aparts" in the near-surface units associated with lateral movement along deep basement faults that subsequently filled with clay and small gravel from above. A number of the clay dikes were examined for shear textures or slickensides, which could suggest lateral (strike-slip) movement. A few minor textures suggesting some limited component of horizontal motion were found, but not enough to make any definitive interpretations.

Faults and clay dikes are evident in the McNairy Formation. Clay dikes are generally absent in the overlying Metropolis materials. Joints are found in both the McNairy and Metropolis formations. Neither joints, faults, nor clay dikes, however, have been found to offset or disturb either of the overlying alluvial units.

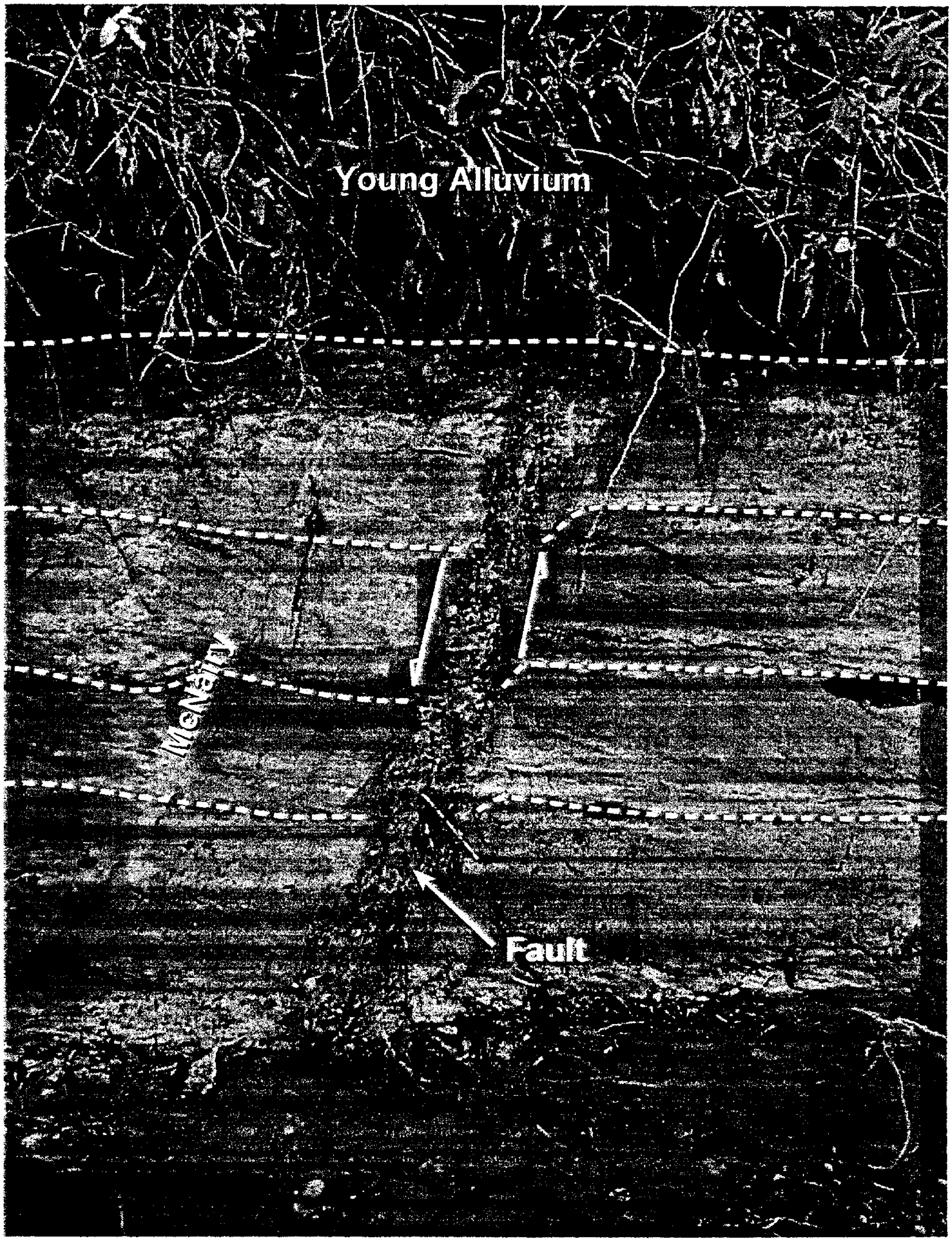
Some of the faults are wide enough to contain a mixture of clay and gravel, while others are single planes with no filling. All of the clay dikes contain the mixture of gray, plastic clay, and small rounded gravel. Geochemical alteration, especially deposition of iron and manganese along the edges of the faults and clay dikes, has enhanced the appearance of many of the structures. Geochemical alteration along joint surfaces is often indicated by light gray coloration.

Figure 6.7 illustrates a clay dike in the McNairy Formation. The gray clay and small white pebbles are clearly evident within the dike, and the boundaries with the McNairy Formation are sharp and distinct. The darker brown bands within the McNairy Formation that parallel the clay dike are iron minerals deposited from groundwater moving through the dike and into the walls of the McNairy. In this example, the clay dike is cleanly truncated at the top by undisturbed gravel layers of the brown lower Metropolis materials. Also the clay and pebbles within the clay dike are distinctly different than the Metropolis materials, suggesting that the clay dike formed and was filled prior to deposition of the Metropolis gravels.

Figure 6.8 illustrates a normal fault in the McNairy Formation. This fault is about 3 inches wide and contains light gray, plastic clay, and small pebbles. The arrows on the figure illustrate the relative movement. Total vertical offset is less than a few inches. The fault terminates at the top of the McNairy Formation. The overlying alluvium is not offset. As illustrated in Figs. 6.7 and 6.8, erosion has removed the Metropolis and old alluvium at these two locations and the young alluvium lies directly on the McNairy Formation.

Figure 6.9 illustrates an example of multiple geologic structures at the same location. In the McNairy Formation, a clay dike about 6 to 8 inches wide is filled with gray clay and small gravel. Groundwater moving through the clay dike has resulted in precipitation of dark red bands of iron oxides on each side of the zone. In this case, the clay dike is truncated at the base of the lower Metropolis gravel. The gravel is continuous and undisturbed across the dike, indicating that formation of the clay dike pre-dated the deposition of the gravel.

Approximately 3 ft of lower Metropolis gravels and 1 ft of finer-grained upper Metropolis materials are exposed above the McNairy Formation at this location. A joint has developed in the Metropolis materials parallel to and aligned with the clay dike. Since the joint breaks the Metropolis materials, it represents deformation after the materials were deposited and at least partially cemented. Note the gray color of the joint in the lower Metropolis. The upward extension of the joint into the finer-grained upper Metropolis is not easily visible on the photo, but the joint clearly extends to the base of the young alluvium where the scientist is pointing his finger. Note that erosion has removed the old alluvium at this location prior to the recent deposition of the young alluvium.



View of south bank of Barnes Creek. Trend of the fault is N52E. The total vertical offset is about 4 inches, down to the east (left). The fault ends at the top of the McNairy Formation and the overlying alluvium is undisturbed.

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Fig. 6.8. Fault at Barnes Creek.

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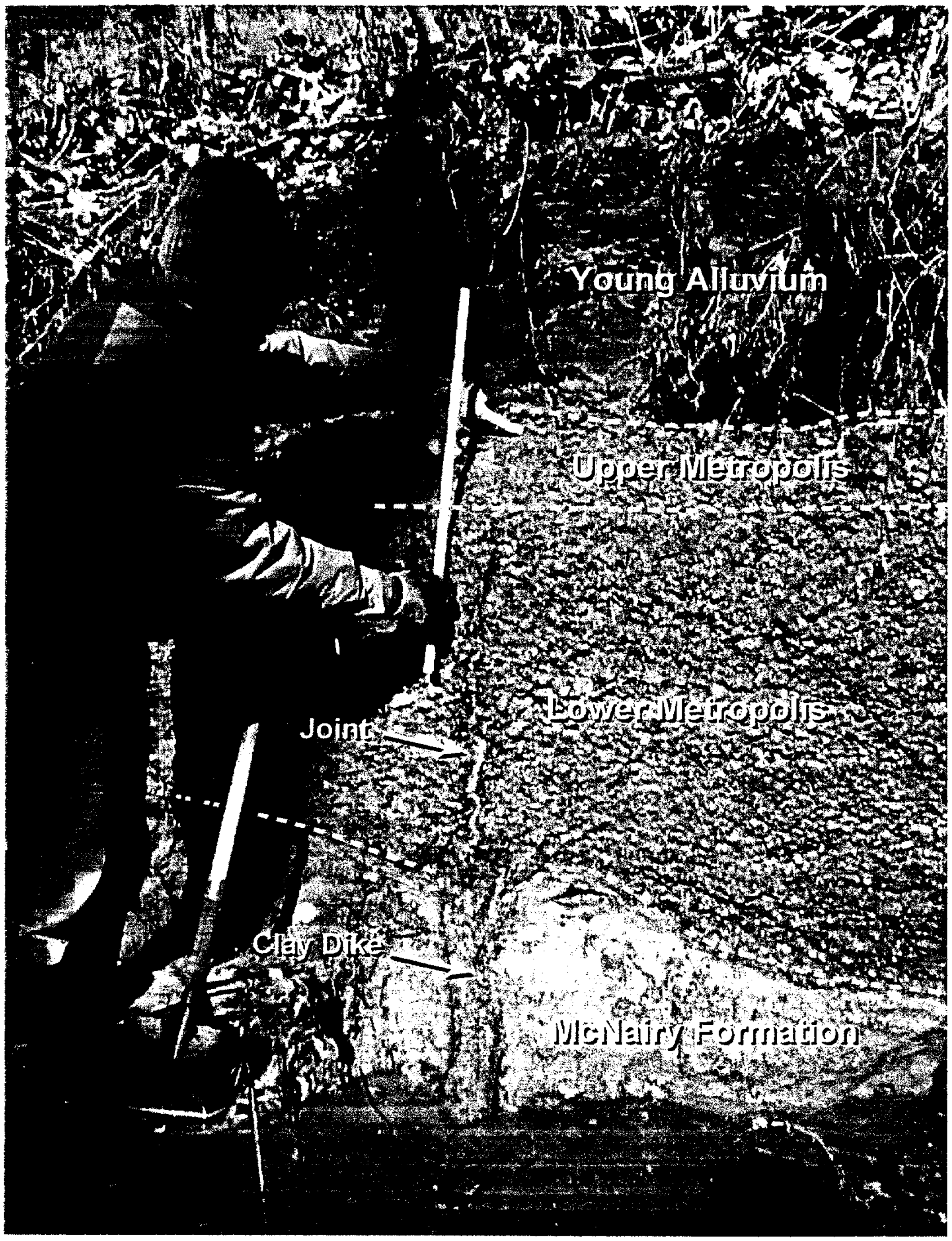

View of north bank of Barnes Creek. Note the 6-in. wide clay dike in the McNairy Formation. The fracture narrows in the lower Metropolis and narrows again in the upper Metropolis, suggesting repeated opening over time.The joint in the Metropolis materials extends through the upper Metropolis to the base of the young alluvium, as pointed to in the photo. The young alluvium is not disturbed.

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Fig. 6.9. Young faulting at Barnes Creek.

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This example illustrates multiple episodes of deformation at the same location, as the original clay dike must be older than the lower Metropolis and the joint must be younger than all the Metropolis materials. This relationship has been observed at several locations within the study area. In most cases, the older deformation is more severe than the youngest (for example old clay dikes associated with younger joints). This observation suggests that multiple episodes of deformation have occurred since the deposition of the lower Metropolis. In general, the intensity of the deformation appears to have decreased with time (i.e., the younger features are generally smaller and show less deformation).

Figure 6.10 illustrates a situation similar to that shown on Fig. 6.9. At this location, a clay dike in the McNairy Formation is truncated by gravels of the lower Metropolis (at point "A" on the photo). However, in this case, reactivation after deposition of the lower Metropolis resulted in development of a normal fault, which offset the lower Metropolis about 8 inches. The normal fault terminates at the base of the young alluvium, shown by the red flagging on the photo.

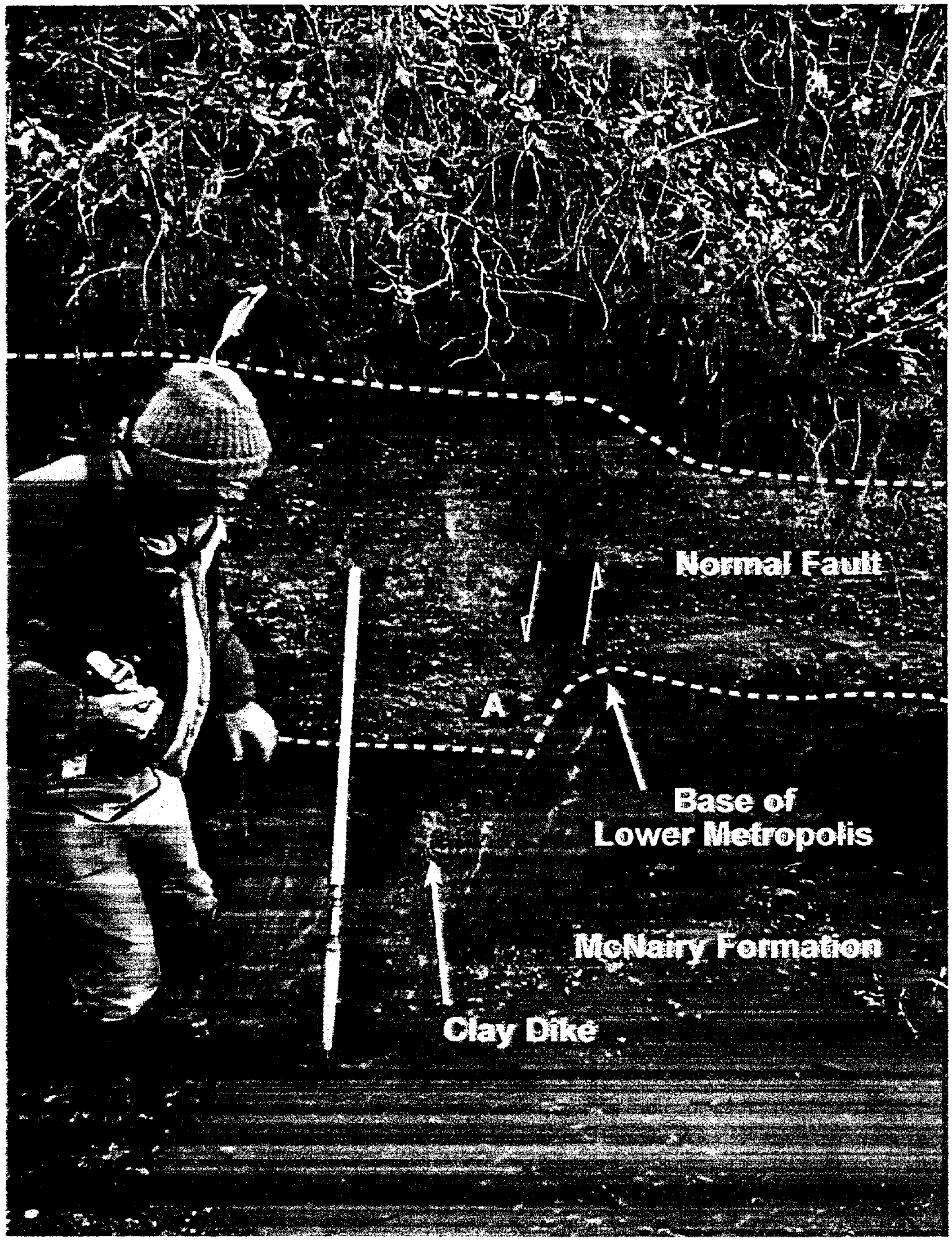
Grabens: Grabens are formed by pairs of normal faults resulting in the down dropping of the central block. All of the grabens exposed in Barnes Creek offset the McNairy Formation and most involve at least a portion of the Metropolis Formation. Most trend to the northeast, roughly parallel to the mapped bedrock faults and the trend of both the NMSZ and WVSZ. Grabens are considered the result of extensional forces. Because this region of the country is under generally east-west compression, the formation of such extensional features is interpreted to be the result of tearing or pulling-apart of near-surface rocks associated with larger-scale strike-slip faulting in the basement rocks.

Figure 6.11 shows one of the grabens in Barnes Creek, which down dropped both the McNairy Formation and the gravels of the lower Metropolis. The graben is approximately 24 inches wide. The materials within the graben have down dropped about 12 inches. The bedding in the lower Metropolis unit within the down-dropped block is intact, suggesting that the graben developed after the gravels were slightly cemented. Joints within the down-dropped block have been bleached a light gray color by geochemical interaction with groundwater. The faults and joints exposed at this location end at the base of the brown young alluvium. The upper Metropolis and old alluvium are not present in this outcrop. The young alluvium is not offset or disturbed. Figures 6.12 and 6.13 provide an additional overview of some of the grabens observed in the creek banks and thin relative position to some of the other types of features observed.

The largest graben in Barnes Creek is the Westerman graben, which has a total width of about 200 ft. A borehole was drilled by Nelson et al. 1997 to a depth of 104 ft without exiting the Metropolis materials. The borehole data indicates at least 75 ft of Metropolis materials in the center of the graben. The Metropolis materials exposed along the stream bank within the graben show a variety of textures, ranging from colluvial "landslide" textures, to normal channel cut and fill structures (Fig. 6.14). This observation suggests possible multiple down dropping episodes during formation of the graben. The Westerman graben is bounded by parallel faults that strike approximately N25°E. At each boundary fault, the McNairy Formation is tilted sharply towards the center of the graben (Fig. 6.15). The central portion of the graben is filled with Metropolis Formation materials.

6.2.3 Age Dating at Barnes Creek

The relative timing of deformations observed in Barnes Creek is determined by the crosscutting of soil layers observed in the field. For example, undisturbed lower Metropolis materials overlie some clay dikes, joints, and faults in the McNairy Formation, indicating that the deformation in the McNairy was completed by the time the Metropolis materials were deposited.



View of south bank of Barnes Creek at station 07+50. Note the clay dike in the McNairy Formation has been truncated by gravels of the lower Metropolis formation at point "A." Later, once at least 4 ft of gravel had been deposited, the right side of the feature reactivated as a normal fault, down dropping the base of the lower Metropolis gravels about 8 inches. The normal fault ends at the boundary with the alluvium, as shown by the square of red flagging. The alluvium is not offset.

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Fig. 6.10. Normal faulting at Barnes Creek.

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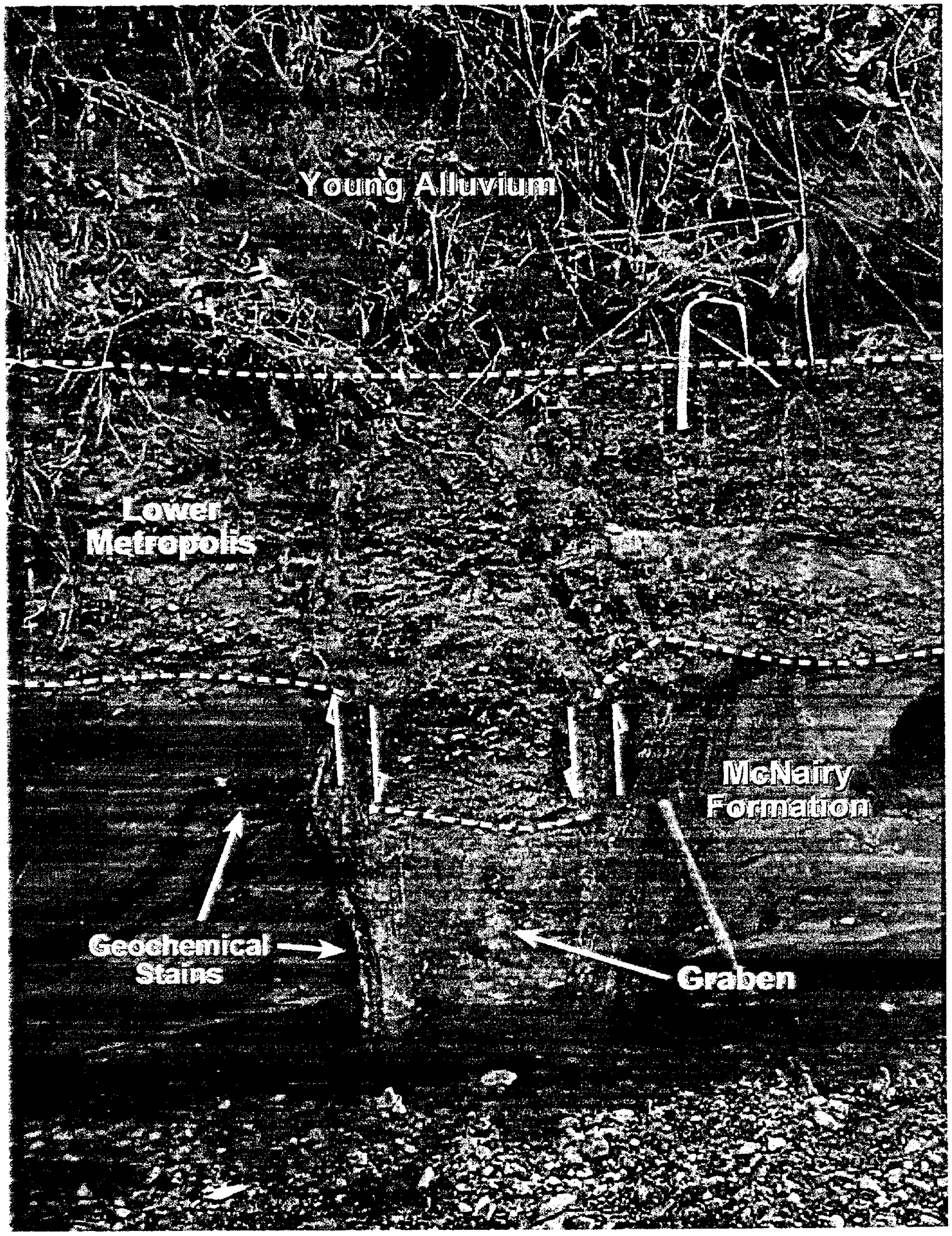

View of south bank of Barnes Creek. The graben is about 24 inches wide and has down dropped the base of the lower Metropolis gravel about 12 inches. Bedding in the Metropolis materials is intact within the graben, indicating the material was cemented to some degree when the graben formed. Numerous joints have been bleached to a light gray color. Several smaller joints extend to the base of the young alluvium. The alluvium is not offset or disturbed.

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Fig. 6.11. Example of graben at Barnes Creek.

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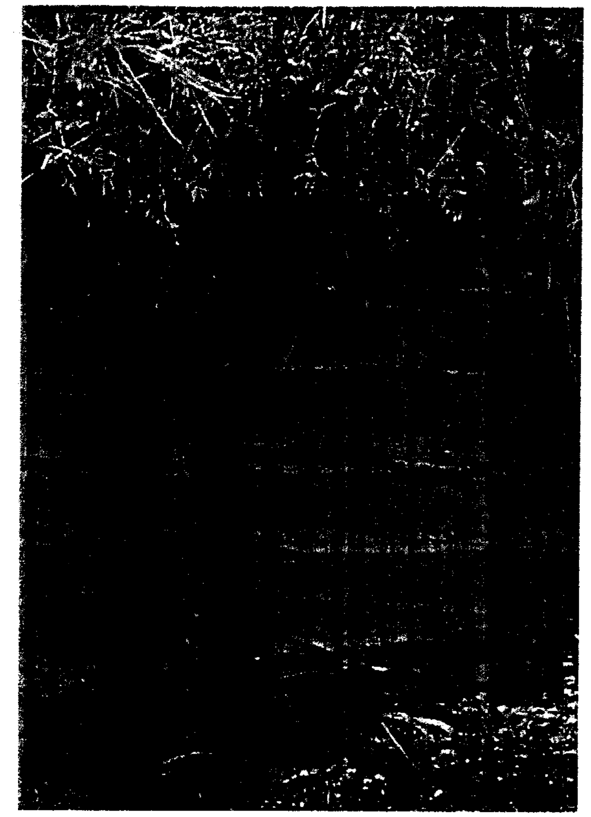

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Fig. 6.12. Faulting and stratigraphy at stops 2 and 3.

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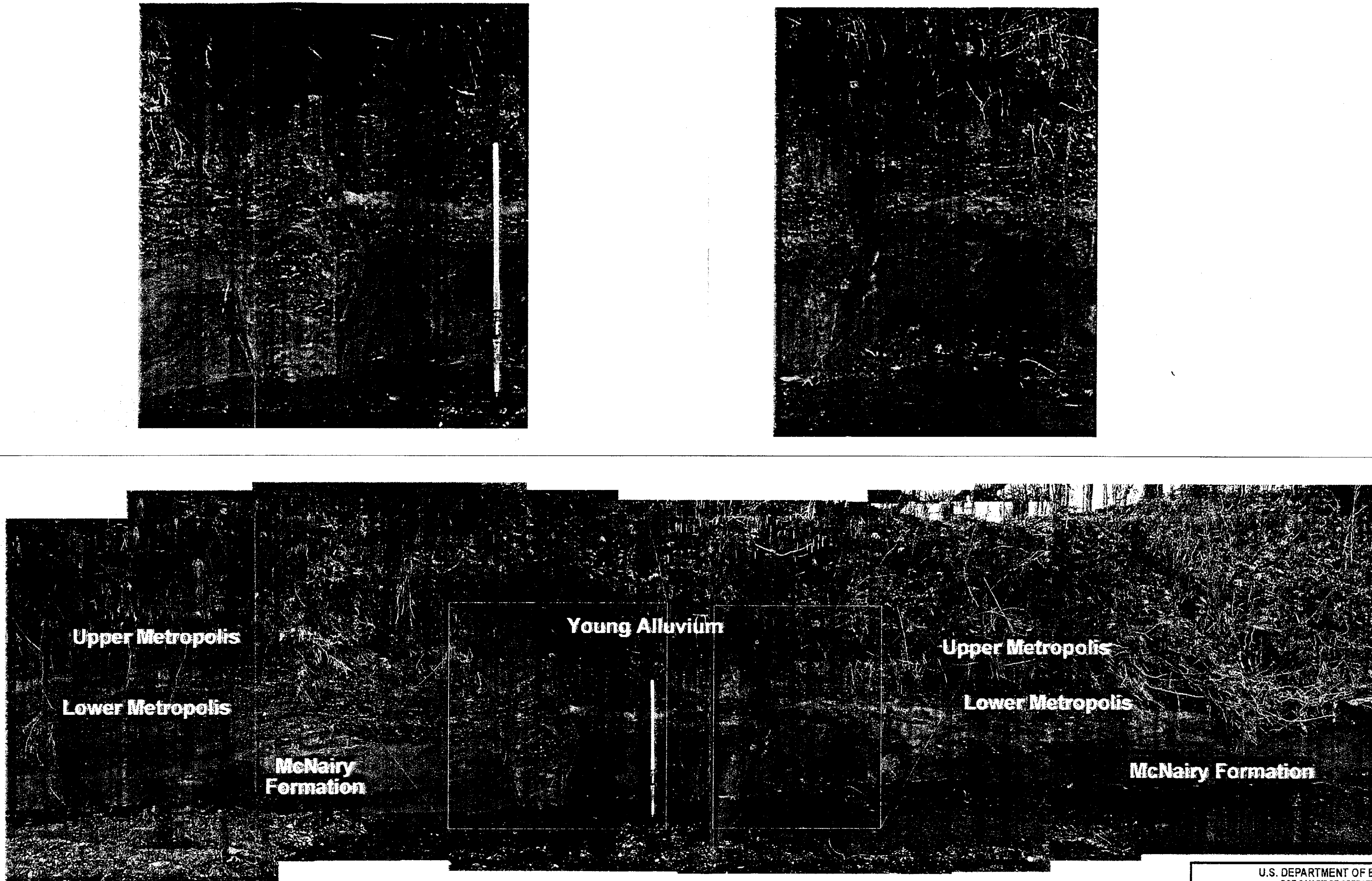

Fig. 6.13. Faulting and stratigraphy at stops 8 and 9.

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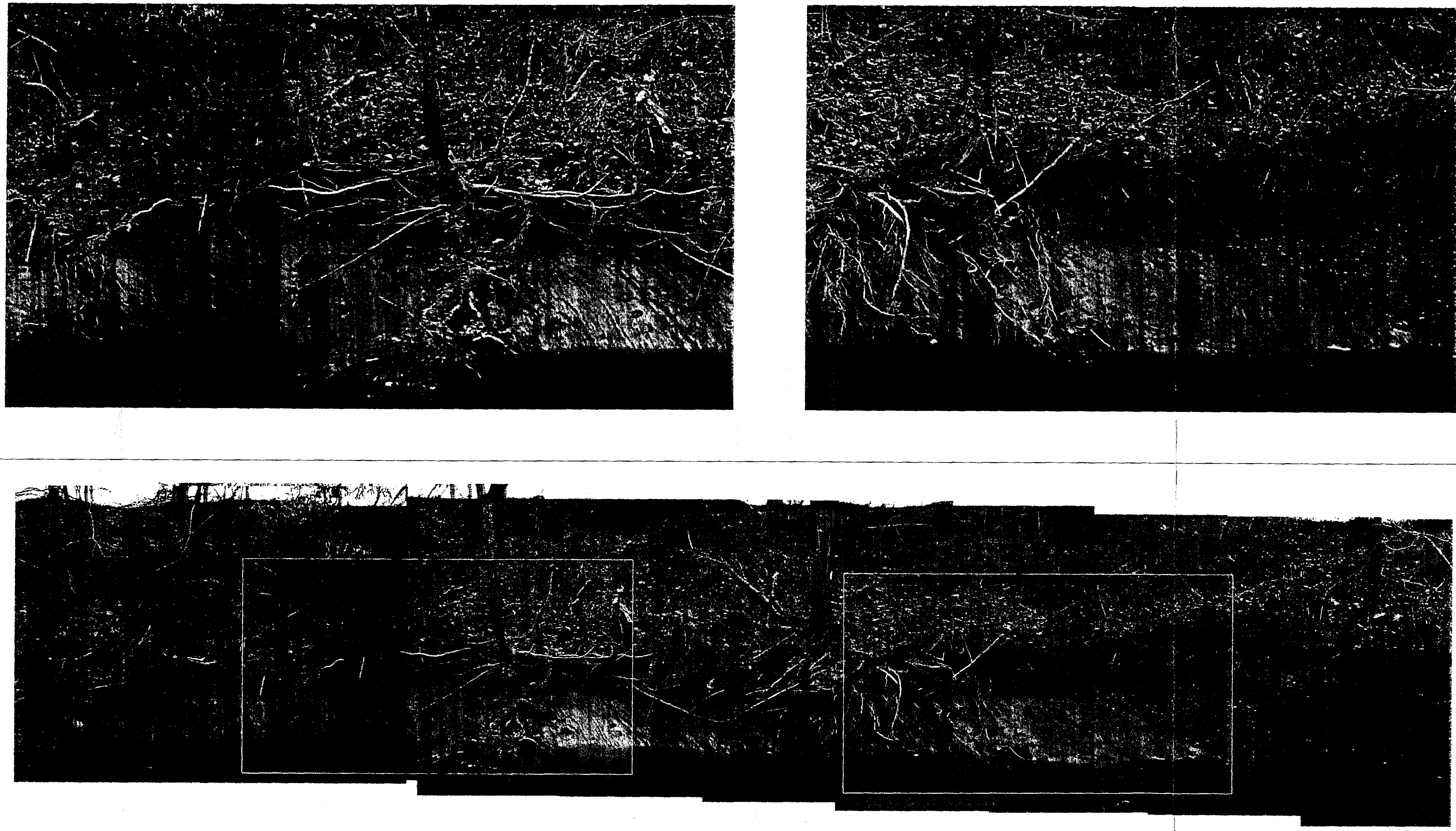

Fig. 6.14. View of the western boundary of the Westerman graben at Stop 12.

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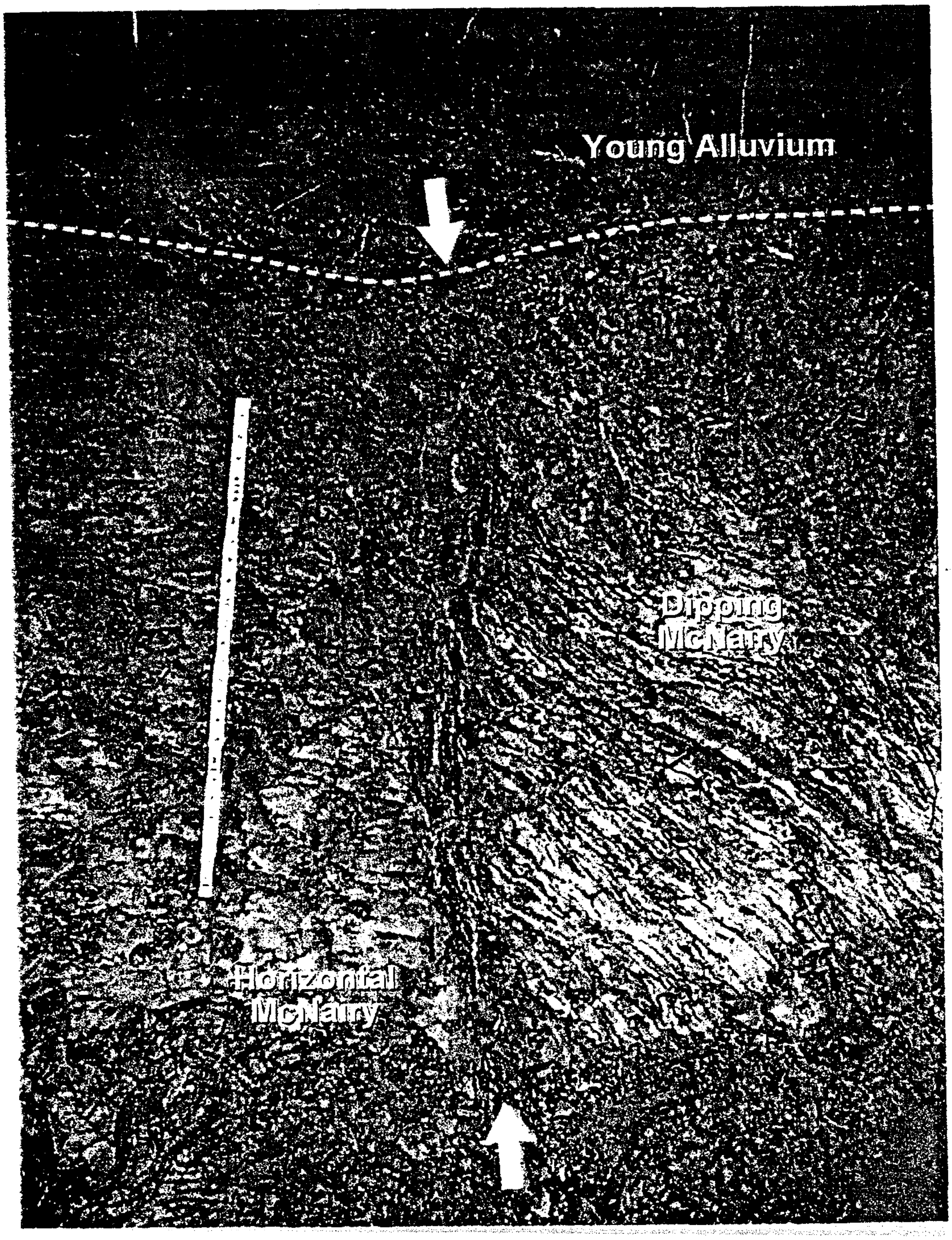

This represents the eastern border of the Westerman graben. To the left of the fault, the McNairy Formation is nearly horizontal. At the fault, the McNairy abruptly dips 30 to 40 degrees to the west, towards the center of the graben. The fault and the dipping beds trend to the northeast. The border fault terminates at the base of the young alluvium.

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Fig. 6.15. Border fault of the Westerman graben.

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In other locations, such as the graben shown on Fig. 6.11, both the McNairy and the lower Metropolis are deformed to the same extent, indicating that the deformation took place after the deposition of both formations. In still other areas, multiple episodes of deformation can be seen (Figs. 6.9 and 6.10), indicating that the deformation took place both before and after deposition of the Metropolis Formation. In no case was either the old, weathered alluvium or the young, unweathered alluvium found to be offset, jointed, or otherwise disturbed.

However, such relative age dating does not provide absolute ages of the deformation. As such, all of the units exposed in Barnes Creek were searched for pieces of detrital charcoal for the purposes of ^{14}C dating. The identification and selection of samples for dating must be done with care to ensure that the sample is a loose piece of charcoal that was deposited at the same time as the surrounding sediments, and not a piece of a much younger root. Where possible, multiple samples are collected from the same locality to provide verification of the ages determined.

A total of 14 charcoal samples were collected in Barnes Creek. Sufficient carbon was present in 10 of these samples. Their radiocarbon ages are presented in Table 6.2. The radiocarbon dates show that the young alluvium is modern (less than 500 years BP). The old alluvium dates from about 500 to 5000 years BP. The upper Metropolis dates from 5000 to 7000 years BP. Because faults were observed to cut this unit, at least one episode of mid-Holocene deformation occurred at Barnes Creek.

Table 6.2. Summary of organic samples and ^{14}C age dating[1] from Barnes Creek bank study

Sample number	Barnes Creek Station	Depth from top of bank (ft)	Stratigraphic location	Measured ^{14}C age (years BP)[2]	Implication
CCFRBS-01	11+82	5.9	Near base of old alluvium	3,630 ± 50*	Minimum age of older gray alluvial materials
CCFRBS-06	13+71	8.5	Young alluvium 3 inches above Metropolis; older alluvium few ft to east	390 ± 50	Approximate age of younger brown alluvial materials
CCFRBS-07	04+61	4.0	6 inches above Metropolis in older gray alluvium	5360 ± 50	Minimum age of upper Metropolis; maximum age of older alluvium
CCFRBS-08	04+61	5.0	0.7 ft into upper Metropolis; possible root cast	5010 ± 50	Minimum age of upper Metropolis
CCFRBS-09	04+61	4.0	In upper Metropolis	6690 ± 50	Age of upper Metropolis
CCFRBS-10	04+61	6.0	Upper Metropolis in stratified gravel	5410 ± 50	Age of upper Metropolis
CCFRBS-11	07+82	6.0	Upper Metropolis in stratified gravel	5700 ± 50*	Minimum age of upper Metropolis
CCFRBS-12	08+21	7.0	Upper Metropolis in stratified gravel	5410 ± 50	Age of upper Metropolis
CCFRBS-13	07+20	3.0	Young alluvium in draw 135 ft north of Barnes Creek	190 ± 50	Minimum age of younger brown alluvial materials
CCFRBS-14	07+20	3.8	Young alluvium	210 ± 50	Minimum age of younger brown alluvial materials

[1]Dates are reported as radiocarbon years before present (BP), where "present" is defined as 1950 A.D.
[2]Measured ^{14}C ages are based on the observed half life of ^{14}C.
*Convential age per Appendix B.

6.2.4 Results of the Barnes Creek Bank Study

Neotectonic studies in a portion of Barnes Creek, Massac County, Illinois, were carried out to determine if mapped faults have moved within the Holocene Epoch. Investigations in the creek identified five geologic units. The three oldest units exhibit faults, clay dikes, and joints. The two younger units are not faulted.

The trends of the geologic structures (generally northeast-southwest) and style of deformation is consistent with bedrock faults mapped to the north of the study area by the ISGS. The northeast-southwest trends are also consistent with the trend of the NMSZ to the southwest, suggesting that these features may be related.

The relative timing of the observed deformations varies. A number of geologic structures are limited to the McNairy Formation and clearly pre-date deposition of the Metropolis materials. Other features involve both the McNairy and Metropolis materials to the same extent, while others appear to be re-activation of old features in the McNairy after or during deposition of the Metropolis materials.

The radiocarbon ages obtained and the geologic relationships observed in the field confirm that repeated deformation has occurred along some of the faults observed in this portion of Barnes Creek. Deformation began prior to the deposition of the lower Metropolis (inferred to be the late Pleistocene), continued during the deposition of the upper Metropolis (which is 5000 to 7000 years old), and most recently occurred in the mid-Holocene, after the deposition of the upper Metropolis (within the last 5000 years). Therefore, faults observed at the Barnes Creek site did extend into Holocene-age deposits. The maximum displacement observed in a single event is approximately 1 ft in the lower Metropolis.

6.2.5 Terrace Graben Investigation

North of Barnes Creek a marshy swale about 300 ft wide trends northeast-southwest, directly in line with a normal fault exposed in the streambed. A small creek formerly flowed through this swale. The creek was re-routed to the west at the same time Barnes Creek was channelized. The elevation in the swale is about the same as the elevation in the broad bottomland. During rains, surface water travels along the swale on its way to Barnes Creek.

Previous investigators (Nelson et al. 1997) have suggested that the observed morphology represents a graben that offsets the Pleistocene terrace, hence the name "terrace graben." If this is a fault-related feature, the observation that it is expressed in the local topography suggests that the motion may be very recent.

Previous drilling on the terrace showed the loess overlying less than 7 ft of Metropolis Formation, resting on the McNairy Formation. A drillhole within the swale penetrated 10 ft of alluvium overlying about 60 ft of Metropolis, then McNairy silt and sand with steeply dipping lamination. Area investigations have found that the Metropolis ordinarily is 5 to 15 ft thick, suggesting faulting or down cutting and subsequent infilling of an old stream channel is responsible for the thicker section beneath the swale.

During the regional Fault Study, a GPR survey was performed along three parallel profiles that cross the postulated graben area. Ten DPT borings were then drilled on the adjoining terrace and across the swale to better define the subsurface and locate the border faults. Organic samples were collected to determine the age of movement (Table 6.3).

Table 6.3. Summary of organic samples and ^{14}C age dating[1] at the terrace graben area

Sample number	Boring	Depth (ft)	Measured ^{14}C age (years BP)[2]	Implication
CCFRD460-1	DPT-460	3.5	1,160 ± 40	Age of alluvium
CCFRD560-1	DPT-560	11	9,160 ± 50	Approximate age of top of Metropolis
CCFRD610-1	DPT-610	5.2	7,230 ± 40	Age of bottom of alluvium or top of Metropolis
CCFRD736-1	DPT-736	43.7	11,130 ± 60	Age of Metropolis
CCFRD736-2	DPT-736	21.8	10,760 ± 50	Age of Metropolis

[1]Dates are reported as radiocarbon years before present (BP), where "present" is defined as 1950 A.D.
[2]Measured ^{14}C ages are based on the observed half life of ^{14}C.

Figure 6.16 presents the results of these investigations. On the terrace, the loess is 7–10 ft thick and mantles the Metropolis. The Metropolis is 10–15 ft thick and overlies the McNairy. Bedding in the McNairy is nearly horizontal in most terrace boreholes. The loess thickness is nearly uniform on the terrace, outside of the graben. The top of the Metropolis Formation slopes nearly parallel with the ground surface.

Within the swale, two areas where the loess is either eroded away and replaced by more recent alluvium or reworked into more recent alluvium were identified. The first location is near station 460 and is about 4 ft thick; the second is between stations 610 and 800 and is up to 10 ft thick. Within the lower area, the Metropolis is present below the loess and alluvium. The Metropolis is generally identified by the first occurrence of chert pebbles in the borehole and includes silty sand, sandy silt, or clay. These sediments are strongly mottled in shades of gray, yellow, and orange.

Between stations 600 and 800, the Metropolis thickens to a maximum of more than 40 ft in DPT 736. The materials within the deeper part of the graben are generally finer grained than the shallower Metropolis sediments. Bedding in the fine grained sediments is horizontal and the layers exhibit a varve-like texture. This texture is usually indicative of a low energy depositional environment, often in a peri-glacial setting.

Investigation of the terrace graben area concluded that the observed stratigraphy is consistent with a combination of two models: (1) a graben with up to 50 ft of displacement within the past 12,000 years, and (2) an erosional feature with up to 50 ft of infilling within the past 12,000 years. The steeply dipping McNairy in the low area tends to support a graben fault model. The character of the deeper deposits, however, suggests an erosional and depositional origin. Therefore, the observed morphology is not necessarily a fault-related feature (graben) as had been previously suspected.

The location of the organic material samples and their corresponding ^{14}C ages are shown on Fig. 6.16. The data indicate that the alluvium was deposited within the past few thousand years. The most important aspect of the ^{14}C ages is that the deep fine-grained sediments beneath the Metropolis are approximately 11,000 years old, indicating that the overlying Metropolis dates from the late Pleistocene or early Holocene. Figure 6.17 shows one of the organic zones that was dated using ^{14}C techniques.

Plans originally called for trenching at one or both of the boundary faults of the terrace graben. The trench would need to be at least 10 ft deep to expose the base of the alluvium and/or loess. Unfortunately, the most likely place to encounter a border of the graben fault is the worst possible place to excavate. The water table is just below the ground surface and the present drainage passes directly through the location where a trench would be most appropriate. In addition, the landowner was generally opposed to opening a large excavation.

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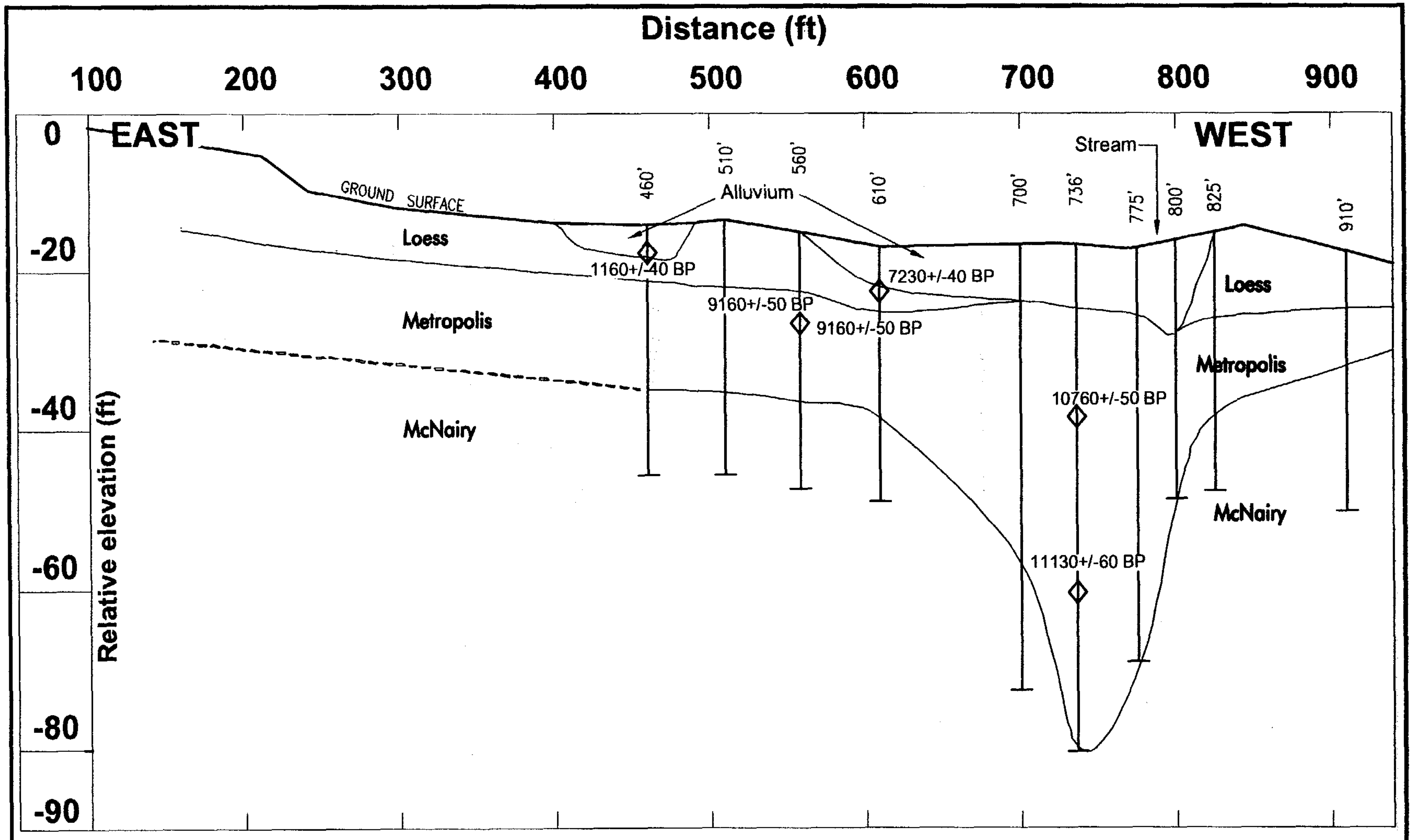

LEGEND:
◇ ...Location of samples for ^{14}C age dating
———...............Interpolated interface
— — —.................Projected interface
BP.................is defined as 1950 A.D. Before Present, where "present"

0 50 100
HORIZ. SCALE: 1"=100'
0 10 20
VERT. SCALE: 1"=20'

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Fig. 6.16. East-west cross section of the terrace graben area.

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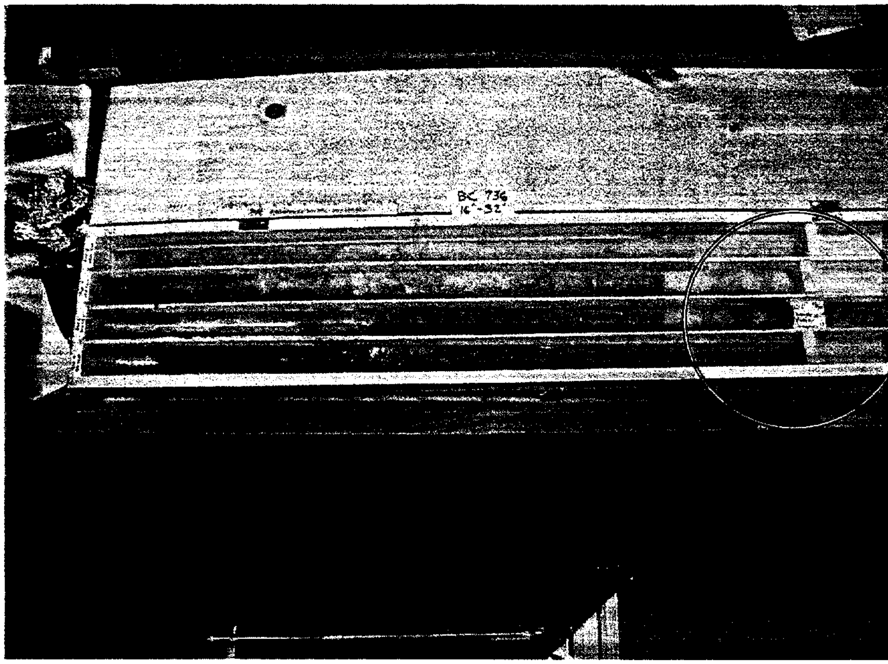
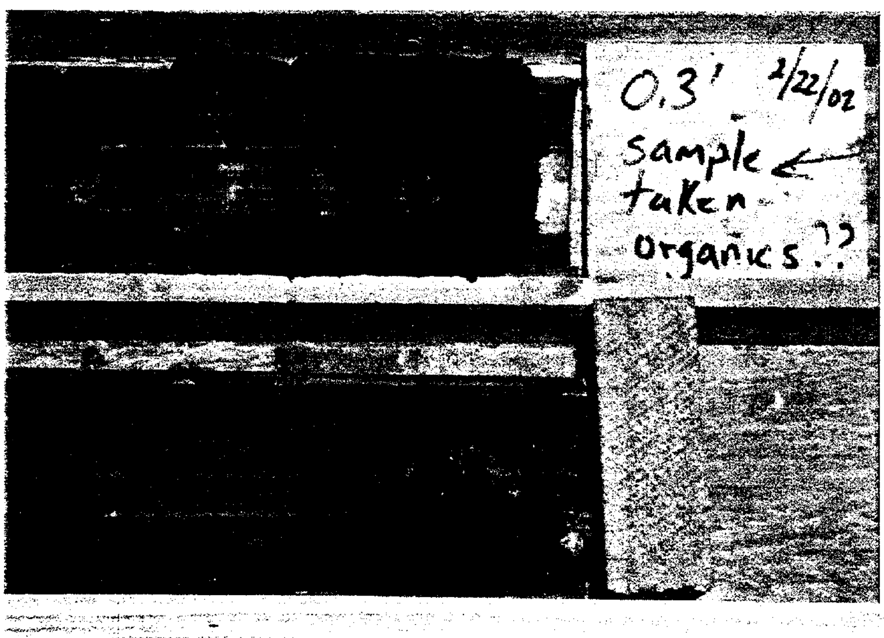


(a). Location of organic sample from the terrace graben area.



(b). Close-up organic sample from the terrace graben area.
(DPT 736 Depth 21.8ft.)

Measured ^{14}C age	10760 ± 50 years BP
Conventional ^{14}C age	10800 ± 50 years BP

BP= before present, where "present" is defined as 1950 A.D.

Fig. 6.17. Organic sample from the terrace graben area.

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Figure No. 6.17
DATE 7/26/02

Based on these factors and the success in obtaining datable materials in Barnes Creek, the DOE investigation team deferred trenching at this site. The investigation at the terrace graben area did provide an age constraint for the lower Metropolis that is present in the banks at Barnes Creek.

6.3 SITE-SPECIFIC FAULT STUDY

The purpose of the site-specific Fault Study was to answer Questions 4 and 5 posed by the Project Core Team; namely, whether there is evidence of Holocene-age displacement at PGDP, and whether there are faults underlying the potential CERCLA waste disposal facility site. The site-specific Fault Study was conducted in two phases, referred to as the initial activities and follow-up activities. The initial activities included a p-wave survey, a GPR calibration survey, and a Project Core Team meeting.

As called for in the Seismic Assessment Plan, the follow-up activities originally included an s-wave survey, a GPR survey, 10 DPT boreholes, and excavation of 3 test pits and a trench.

Based on the results of the GPR calibration survey conducted during the initial activities, it was concluded that GPR was not a viable investigative tool at Site 3A. The local soils prevented the effective penetration of the radar waves. Consequently, the follow-up GPR survey was not implemented at Site 3A. Further details regarding the GPR calibration survey for the Fault Study are presented in Chap. 2 and in Appendixes C and D.

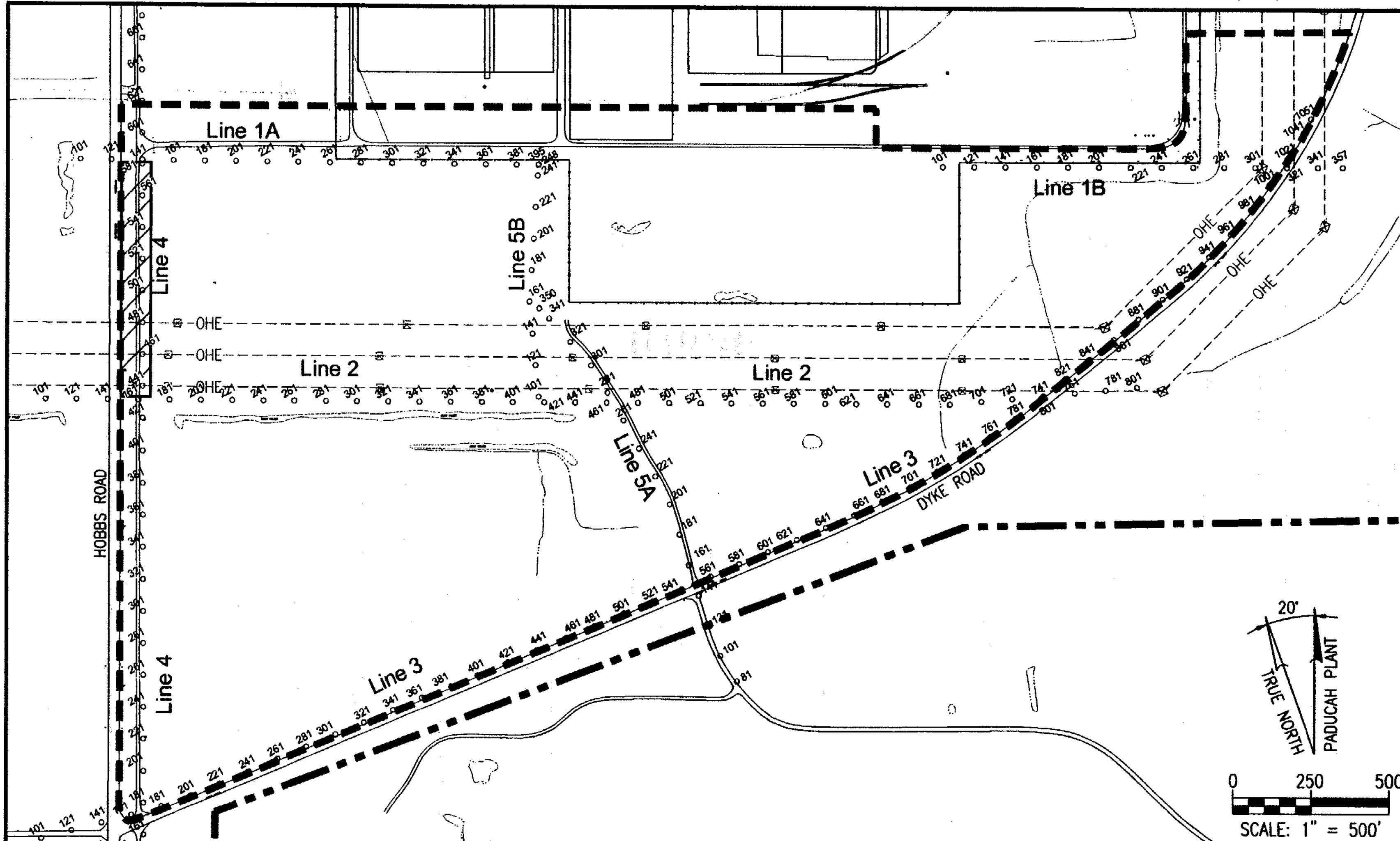
Although excavation of test pits and a trench were planned to collect visible evidence of shallow faulting, the locations of the suspected faults defined during the initial activities resulted in field conditions that would greatly hinder excavation. Several factors led the DOE investigation team to defer trenching. These factors included the following:

- The depth and size of the excavation that would be required to reach "marker" units that could be surveyed for evidence of faulting would be excessive (greater than 20 ft deep).
- The physical characteristics of the loess deposits suggest that holding open an excavation in these materials would be difficult and present a safety concern for the investigators.
- Relatively shallow water levels in the loess deposits at the potential trenching locations would have made excessive dewatering a requirement.
- The presence of potential wetlands, paved roads, and an underground utility immediately adjacent to a potential test pit and trench location would create further obstacles.

The DOE investigative team concluded that if additional data were to be required, a safer approach might include the installation of tightly-spaced DPT borings in lieu of test pits and trenches.

6.3.1 Initial P-wave Survey

During the initial activities, p-wave seismic reflection data were acquired along seven survey lines totaling approximately 16,000 lin. ft of surface coverage. The target zone for the p-wave survey extends from the bedrock surface (approximately 400 ft bgs) upward into the overlying McNairy and Porters Creek Clay Formations (approximately 50 ft bgs). The locations of the survey lines relative to PGDP and other permanent geographic features are shown in Fig. 6.18. Appendix C summarizes the data acquisition and field methods used to conduct the investigation, and includes sections on data processing, interpretation, and conclusions.



LEGEND:

- Asphalt road
- Gravel road
- PGDP boundary
- Site 3A boundary
- Railroad tracks
- Fence line
- Source test area line 4 Shot points 438–581
- Stream
- Electrical utility lines
- O501 P-wave survey location

Fig. 6.18. P-wave survey line locations at Site 3A.

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Figure No. /99049/DWGS/P34SK525
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Completion of the p-wave survey was planned as a key decision point in the site-specific Fault Study. If the initial p-wave survey found no indication of deformation in the sediments overlying the bedrock, then it would be assumed that no young faulting is present and no follow-up activities would be necessary. Conversely, if deformation of the overlying sediments (especially the Porters Creek Clay) were indicated, then additional follow-up activities (including an s-wave survey) would be conducted to determine if the deformation extends up into the even younger (near-surface) loess.

The p-wave survey was successful in imaging several horizons beneath Site 3A, including the top of limestone bedrock, top of the McNairy, and portions of the Porters Creek Clay. A total of 11 potential north-northeast trending faults have been interpreted in the data (Fig. 6.19). Profiles of the instantaneous phase sections from p-wave survey Lines 2 and 3, showing soil borings and potential faults are presented in Figs. 6.20 and 6.21. A discussion of the seismic profiles and their associated potential faults is provided below.

All of the faults are interpreted to trend north-northeast, as shown on Fig. 6.19. All of the fault features are interpreted to dip toward the east-southeast and to extend from the Paleozoic limestone bedrock at least up into the McNairy Formation.

Relative movement on each side of the fault has been depicted as up-thrown (U) or down-dropped (D) on Fig. 6.19. This relative movement along each of the interpreted faults from Fault 1 through Fault 7 is down on the east; representing a series of westward-rotated blocks throughout the western portion of Site 3A. Antithetic faults (ones that dip in the opposite direction of the main normal faults) may be present at Faults 3, 4, and 5. In addition, between Faults 4 and 6, a significant upward-arching (anticline-like) feature is evident as an approximately 10 to 15 msec pull-up in the data (15 to 25-ft vertical rise in soil horizons), as can be seen on Figs. 6.20 and 6.21. This feature is interpreted to be narrower at the northern edge of Site 3A, and to broaden toward the south-southwest. Relative movement from Faults 7 through 11 represents a complex of horst and graben structures throughout the eastern portion of Site 3A, with the graben structures interpreted between Faults 7 and 8 and between Faults 9 and 10.

- **Fault 1:** Fault 1 is identified in the extreme northwest corner of Site 3A. This fault extends from bedrock into the McNairy Formation. Although anomalous reflections are evident on survey Line 1A (Appendix C) above approximately 60 msec (i.e., approximately 140 ft bgs, near the top of the upper sand facies of the McNairy Formation), correlations could not be determined with any degree of certainty.

- **Fault 2:** Fault 2 may represent a series of faults associated with a horst and graben complex in the western and southwestern corner of Site 3A. The northernmost component of Fault 2 is interpreted to offset the top of the McNairy and possibly extend upward into the Porters Creek Clay. Along the southernmost component of Fault 2 along survey Line 4 (Appendix C), a significant discontinuity in the bedrock reflector exists. The character of this reflector, combined with multiple strong diffractions evident beneath the reflector, indicates a significant fault or fracture zone may exist at this location.

- **Fault 3:** Fault 3 extends into the McNairy Formation, and possibly well into or through the Porters Creek Clay. Similar to Fault 1, anomalous reflections at 60 msec (140 ft bgs) are uncertain. Branching features of Fault 3 may include antithetic faults or may be associated with flower structures.

- **Fault 4:** Fault 4 extends well into the McNairy Formation and possibly into or through the Porters Creek Clay. Along survey Line 3 between Fault 4 and Fault 5 (Fig. 6.21), reflections representing the top of bedrock are of high amplitude, but discontinuous, which differs from the relatively coherent reflections from bedrock along most other portions of the line. These characteristics, combined with multiple strong diffractions evident beneath the bedrock reflection, indicate a significant fault or fracture zone may exist at this location.

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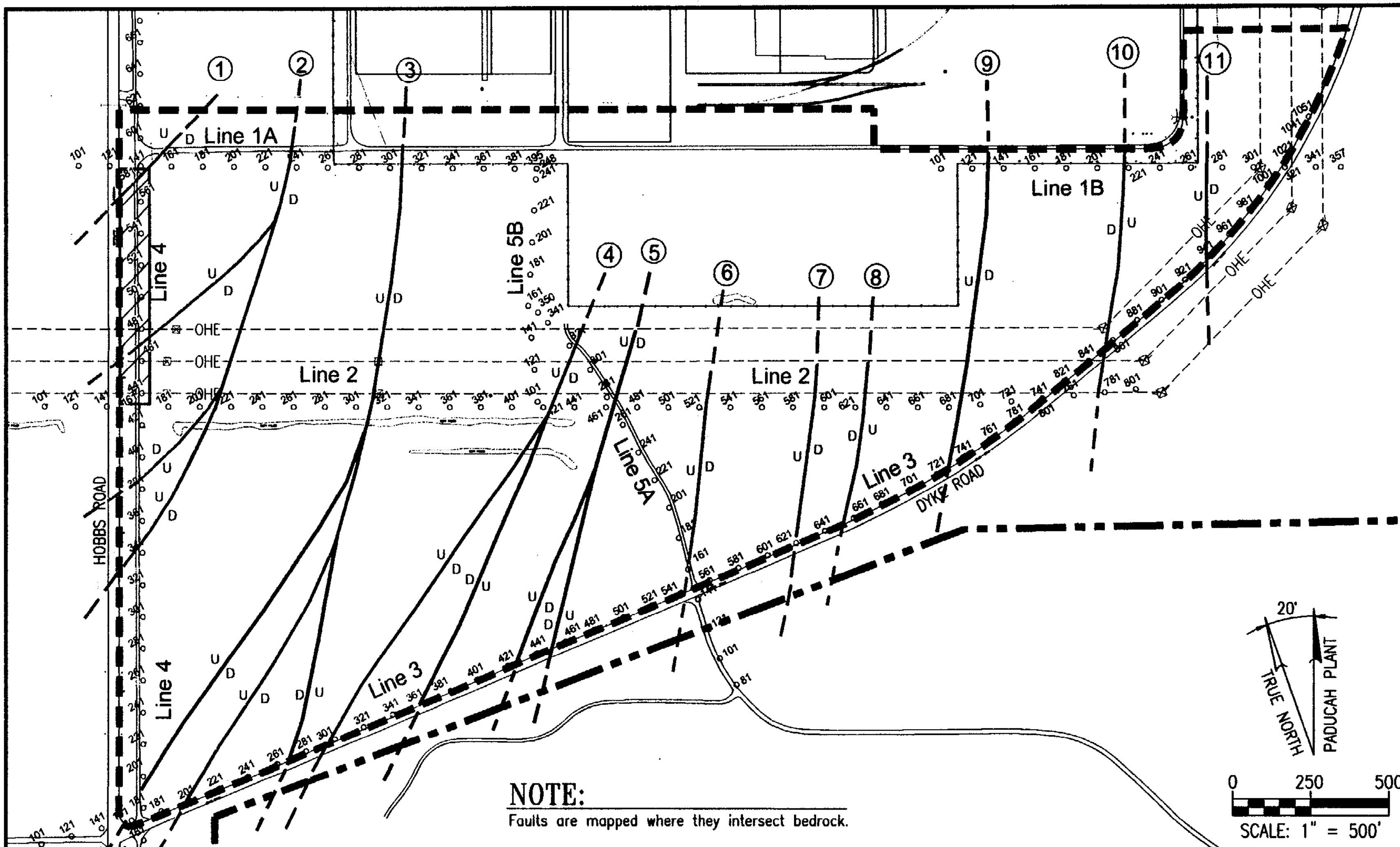

LEGEND:

- Asphalt road
- Gravel road
- PGDP boundary
- Site 3A boundary
- Railroad tracks
- Fence line
- Source test area line 4
- Shot points 438–581
- Stream
- Electrical utility lines
- P-wave survey location
- Interpreted fault location (bedrock and unconsolidated sediments)
- Interpreted fault location (bedrock and lower McNairy formation only)
- U/D Relative movement along fault plane

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Fig. 6.19. Fault interpretation from p-wave survey.

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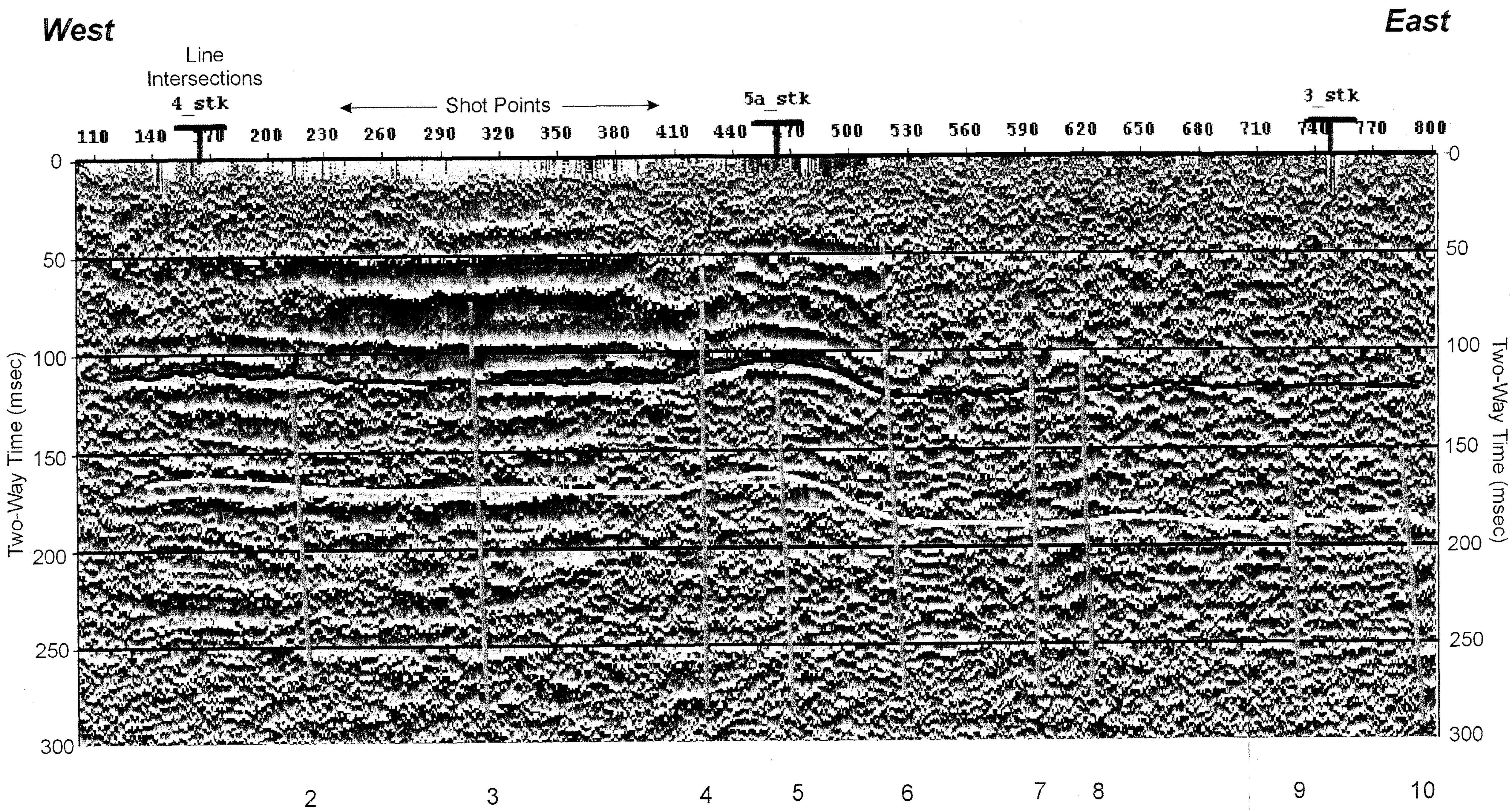

LEGEND
Interpreted top of McNairy Formation (lower sand facies)
Interpreted top of limestone
Interpreted fault

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Fig. 6.20. Line 2 interpreted instantaneous phase section.

Figure No. 6.20
DATE 7/3/02

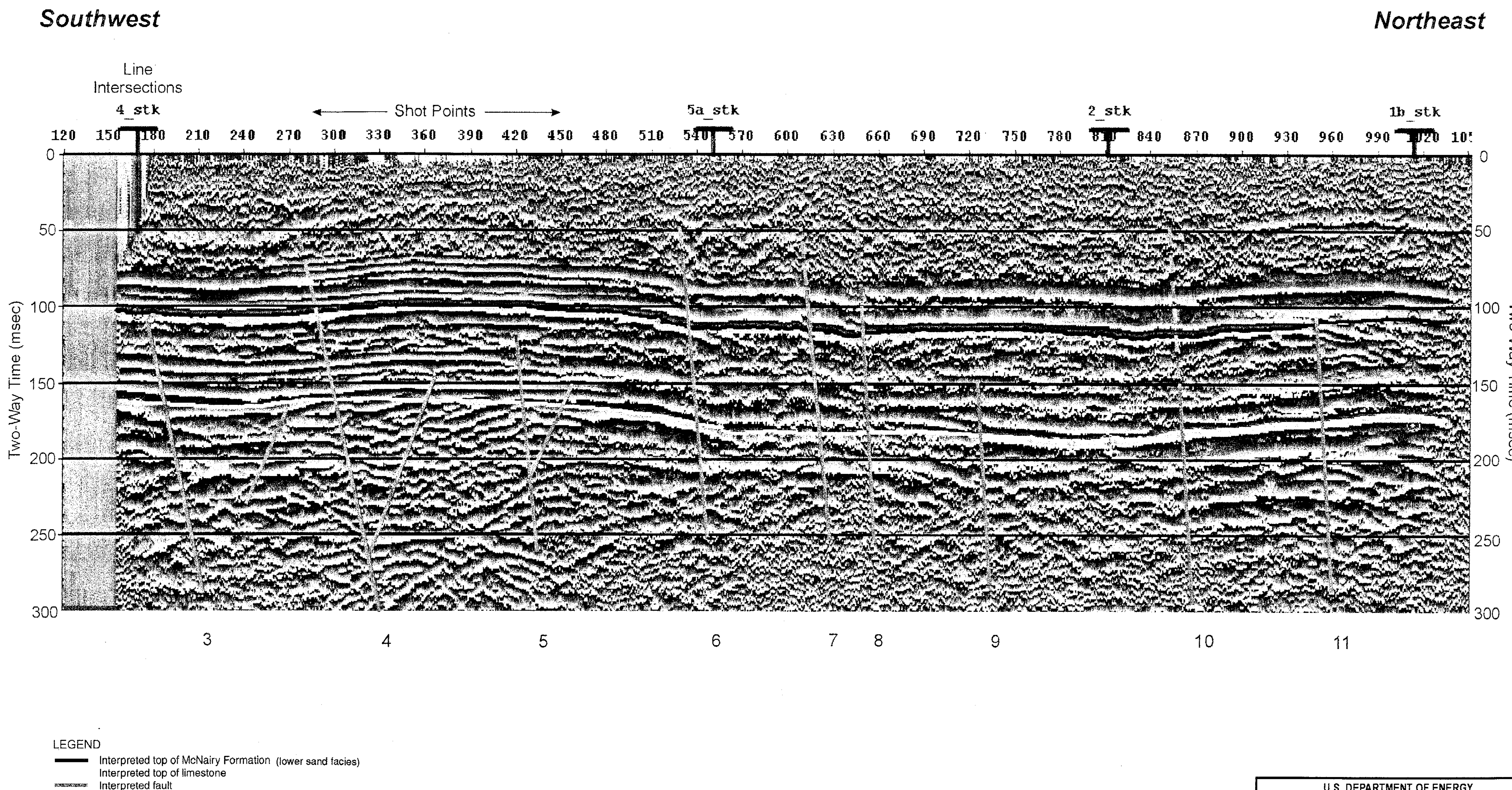

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Fig. 6.21. Line 3 interpreted instantaneous phase section.

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- **Fault 5:** Fault 5 is interpreted as extending into, but not through the McNairy. Similar to Fault 3, branching features may include antithetic faults or flower structures. The high amplitude reflections and multiple strong diffractions beneath the bedrock indicate a significant fault or fracture zone at this location.

- **Fault 6:** Fault 6 is interpreted to offset the top of the McNairy and possibly extend upward well into or through the Porters Creek Clay. It is associated with the anticline-like feature in the middle of Site 3A.

- **Faults 7 and 8:** Faults 7 and 8 are similar normal faults that are interpreted to extend to the top of the McNairy and possibly into the Porters Creek Clay. The paired faults are interpreted to represent a graben feature between them.

- **Faults 9 and 10:** Faults 9 and 10 are paired faults that, like Faults 7 and 8, are interpreted to represent a graben feature between them. Fault 9 is interpreted to extend through the Paleozoic bedrock, but not up through the McNairy. Fault 10 may extend upward well into or through the Porters Creek Clay, as shown on survey Line 3 in Fig. 6.21.

- **Fault 11:** Fault 11 is located in the extreme northeast corner of Site 3A. It is interpreted as extending into the McNairy Formation. However, some anomalous reflections are evident in survey Line 1B (Appendix C) above approximately 70 msec (i.e., approximately 160 ft bgs, near the top of the upper sand facies of the NcNairy), so that this fault may extend up into the Porters Creek Clay.

All 11 interpreted faults show disruptions near the top of the bedrock limestone and appear to offset that unit. Nine of the 11 faults are interpreted to extend upward into the Cretaceous-age McNairy Formation. Several of these features may extend well into or possibly through the Paleocene-age Porters Creek Clay Formation (Fig. 6.19).

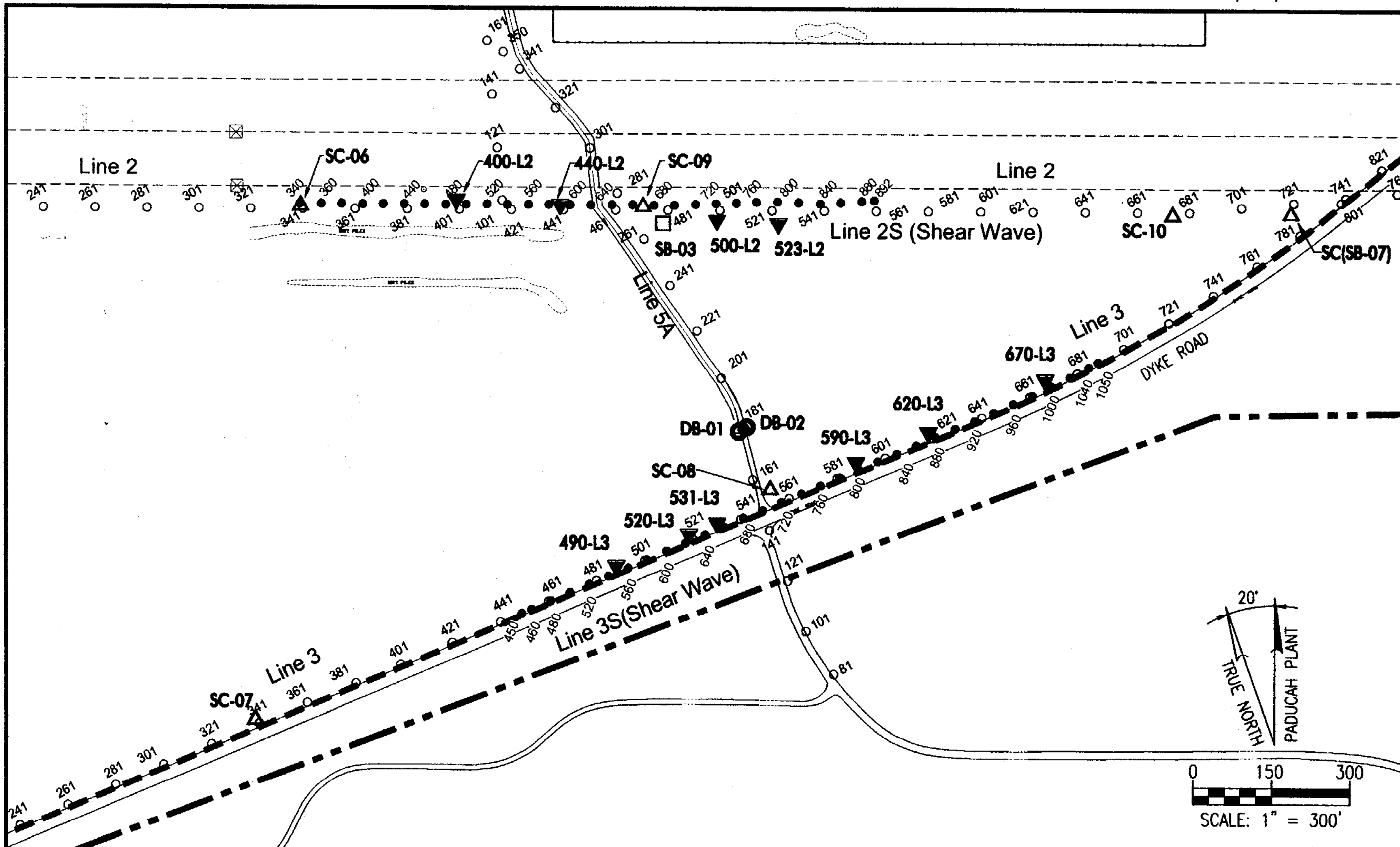
It is important to stress that the p-wave survey was not intended to have sufficient resolution to determine if postulated faulting extends into the Terrace Deposits and/or loess that overlie the Porters Creek Clay. This would require follow-up activities including a more focused s-wave survey.

6.3.2 Follow-up S-wave Survey and Associated DPT Borings

Following the interpretation of the p-wave survey data, a Project Core Team meeting was held on January 15, 2002 to reach consensus on the results, select the location for the planned s-wave survey, and review the remainder of the Site 3A activities. The Project Core Team concluded that the s-wave survey should focus on the interpreted faults that appear to extend upward into the shallow sediments overlying bedrock, occur adjacent to or bound the anticline-like feature, and trend through the central portion of the site (PPC 2002). The s-wave survey sections would have approximately twice the vertical resolution and 2.5 times the horizontal resolution of the p-wave survey sections, but would only be used to image the shallower portions of the subsurface.

The s-wave survey was conducted along two survey lines (Lines 2S and 3S) totaling approximately 2300 lin ft. The locations of the s-wave survey Lines 2S and 3S relative to p-wave survey Lines 2 and 3 are presented as Fig. 6.22.

Figure 6.22 also shows the locations of 10 DPT boreholes that were drilled along the same s-wave survey lines. The locations of nearby soil borings and SCPT soundings are also shown for reference. These boreholes and soundings provided information on the stratigraphy and were used in interpreting the seismic survey data. The SCPT soundings provided actual shear-wave velocities at known locations and



LEGEND:

- ....Asphalt road
- ....Gravel road
- ....PGDP boundary
- ....Site 3A boundary
- ....Fence line
- ....Stream
- ....DPT location
- ....Deep boring location
- ....Shallow boring location
- ....SCPT location
- — —OHE— — ....Electrical utility lines
- ●401......... S-wave survey stations
- ○501......... P-wave survey stations

Fig. 6.22. S-wave and p-wave survey line locations at Site 3A.

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depths. The DPT boreholes provided soil cores to correlate with and help interpret the seismic data. In addition, organic samples were collected from these cores to provide age constraints.

The s-wave survey, together with the associated DPT boreholes, was successful in imaging several near-surface horizons and faults beneath Site 3A. Horizons evident in the s-wave survey data and confirmed by the DPT boreholes, SCPT soundings, and soil borings include the near-surface loess, a firm sand unit underlying the loess, and the Porters Creek Clay. These horizons can be seen on Figs. 6.23 and 6.24. Each of these horizons is discussed below.

Loess: The shallowest reflectors on Lines 2S and 3S occur between 90 and 130 msec (approximately 12 to 25 ft bgs), as shown on Figs. 6.23 and 6.24. The base of the shallowest reflectors has been highlighted in yellow. Based on DPT soil cores, the horizon is deemed to be near the base of the loess at Site 3A. The shallowest bright reflector on Line 2S (Fig. 6.23) exists only on the east side of the section from shotpoints 670–890. There is no significant information in the DPT data to indicate changes in material properties that would produce this reflector. The DPT data from shot points 741 and 798 indicate that some gravels are present about 16–24 ft bgs, but they appear to be minimal. It is possible that some other lithologic character, such as clay content, is affecting soil "stiffness" to produce these reflectors.

On Line 3S (Fig. 6.24), a series of high amplitude peaks and troughs extend across the top of the section. This package of reflectors varies laterally in thickness and appears to define near surface channel features on the east side of the section. It is not obvious from the DPT data what lithologic changes might be causing the reflectors observed along the top of this line. However, most of the DPT data along the line indicates the presence of sand and/or gravel layers on the order of 17–23 ft bgs. Thin, coarse-grained layers at these depths would likely produce the shallow reflectors observed on the sections.

Firm Sand: The SCPT and DPT data document the existence of a sand layer at approximately 30-35 ft bgs. This firm sand (highlighted in blue on Figs. 6.23 and 6.24) produces a strong reflector on the s-wave survey sections at approximately 150 msec (approximately 32 ft bgs). The firm sand reflector is certainly the dominant feature on Line 2S. On Line 3S, the firm sand manifests itself as a package of bright reflectors across the central part of the section, fading somewhat at both ends. DPT information along Line 3S indicates that the firm sand is not a single unit here, but represented by a series of interbedded sands and clays. The firm sand may represent channel or meander loop sedimentation, hence it might be expected to exhibit rapid lateral variations in character.

Porters Creek Clay Formation: The top of the Porters Creek clay unit is fairly well defined by intrusive testing. DPT boreholes, SCPT soundings, and mud rotary boreholes extend down to the top of the Porters Creek clay within Site 3A, and this information has helped to identify the horizon on the seismic sections. To help confirm these findings, the s-wave travel time to the horizon picked as the top of the Porters Creek clay was confirmed. The calculation, which assumes an average s-wave velocity of 700 ft/sec and depth to the Porters Creek clay of 50 ft, places the reflector at roughly 200 msec.

The intrusive information (DPT, SPT, and SCPT) indicates that the reflector seen on the s-wave survey sections is actually a gravelly sand layer directly overlying the Porters Creek Clay. SCPT data show a large increase in tip resistance and shear stress within this gravelly sand, and these properties are directly related to shear-wave velocity. It appears that both the top and the bottom of the gravelly sand are being imaged on the s-wave survey sections, as evidenced by the peak-trough-peak sequence (blue-red-blue) seen on Line 3S. Because the top of the Porters Creek Clay corresponds to the bottom of the gravelly sand, the lower peak has been picked on the seismic sections.

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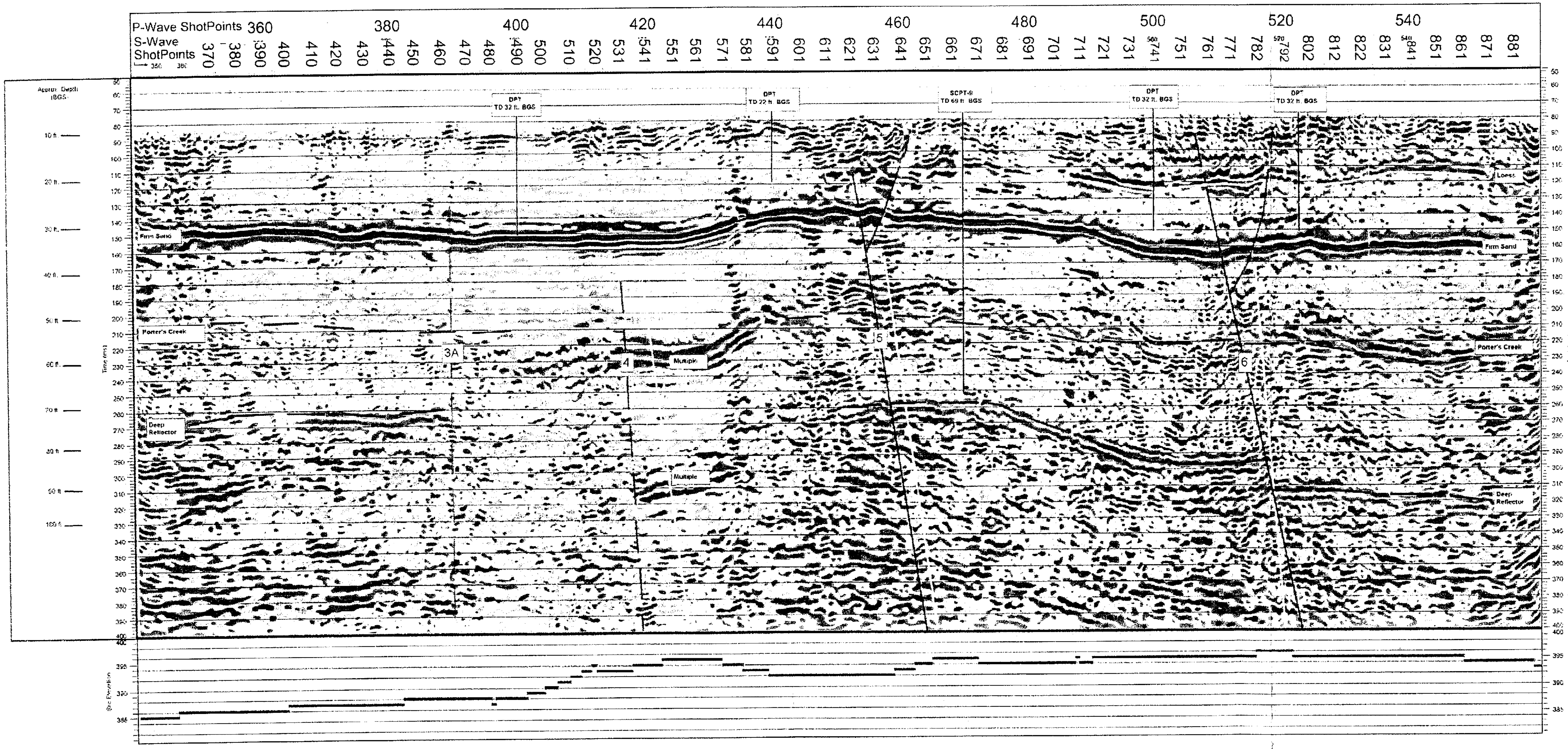

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Fig. 6.23. Migrated section for s-wave survey Line 2S.

Figure No. 6.23
DATE 7/3/02

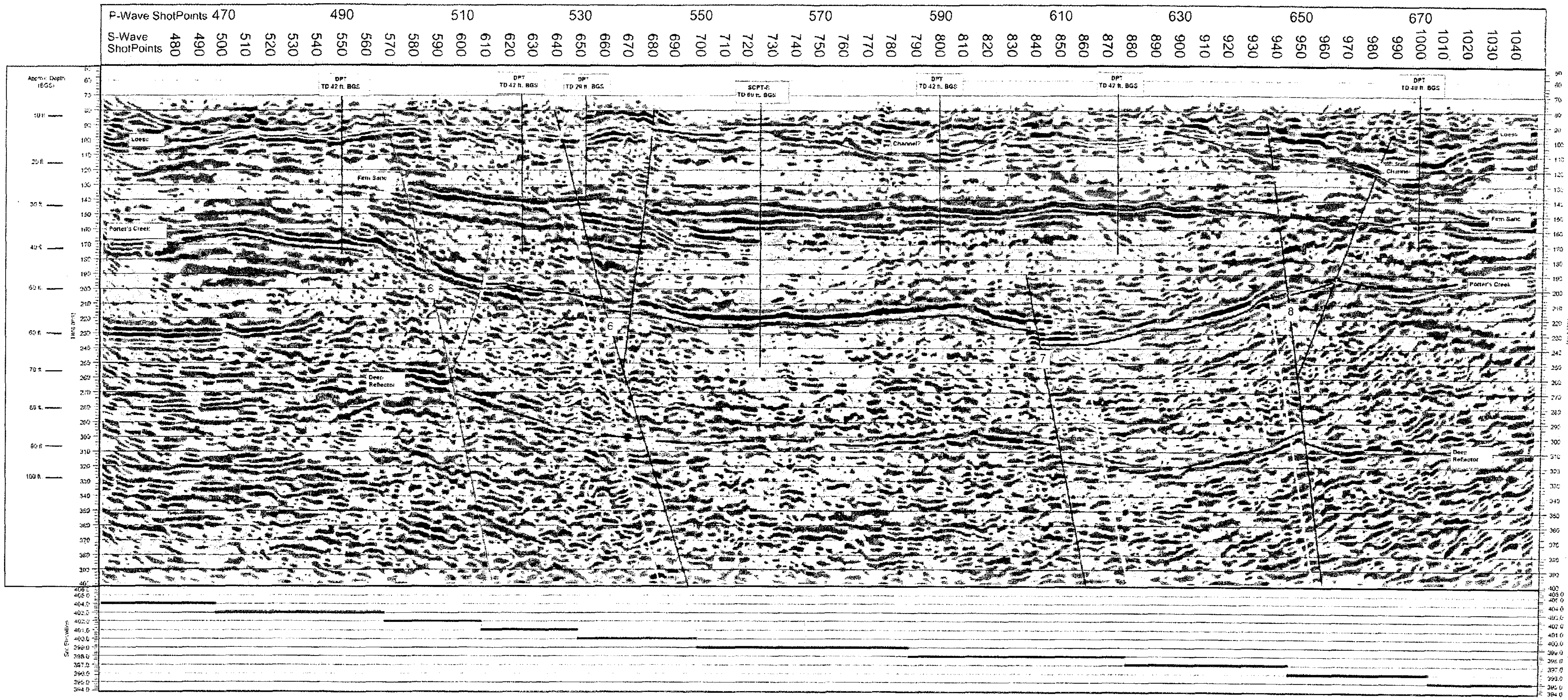

Fig. 6.24. Migrated section for s-wave survey Line 3S.

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The top of the Porters Creek Clay reflector is easily traced across the entirety of Line 3S (Fig. 6.24). On Line 2S, the top of Porters Creek Clay reflector is only evident on the eastern side of the line (Fig. 6.23). Moving west from shot point 785, the reflector gradually weakens and then disappears altogether. SCPT sounding SC-09 provides a possible explanation for this occurrence. At this location, the gravelly sand has bifurcated into two thin layers, each less than 2 ft thick. SCPT tip resistance within these gravelly sands remains high, although shear stress at SC-09 shows less contrast than at SC-08 on Line 3S. It is, therefore, likely that the gradual disappearance of the reflector as one moves west along Line 2S is at least partially because of the thinning and splitting of the gravelly sand.

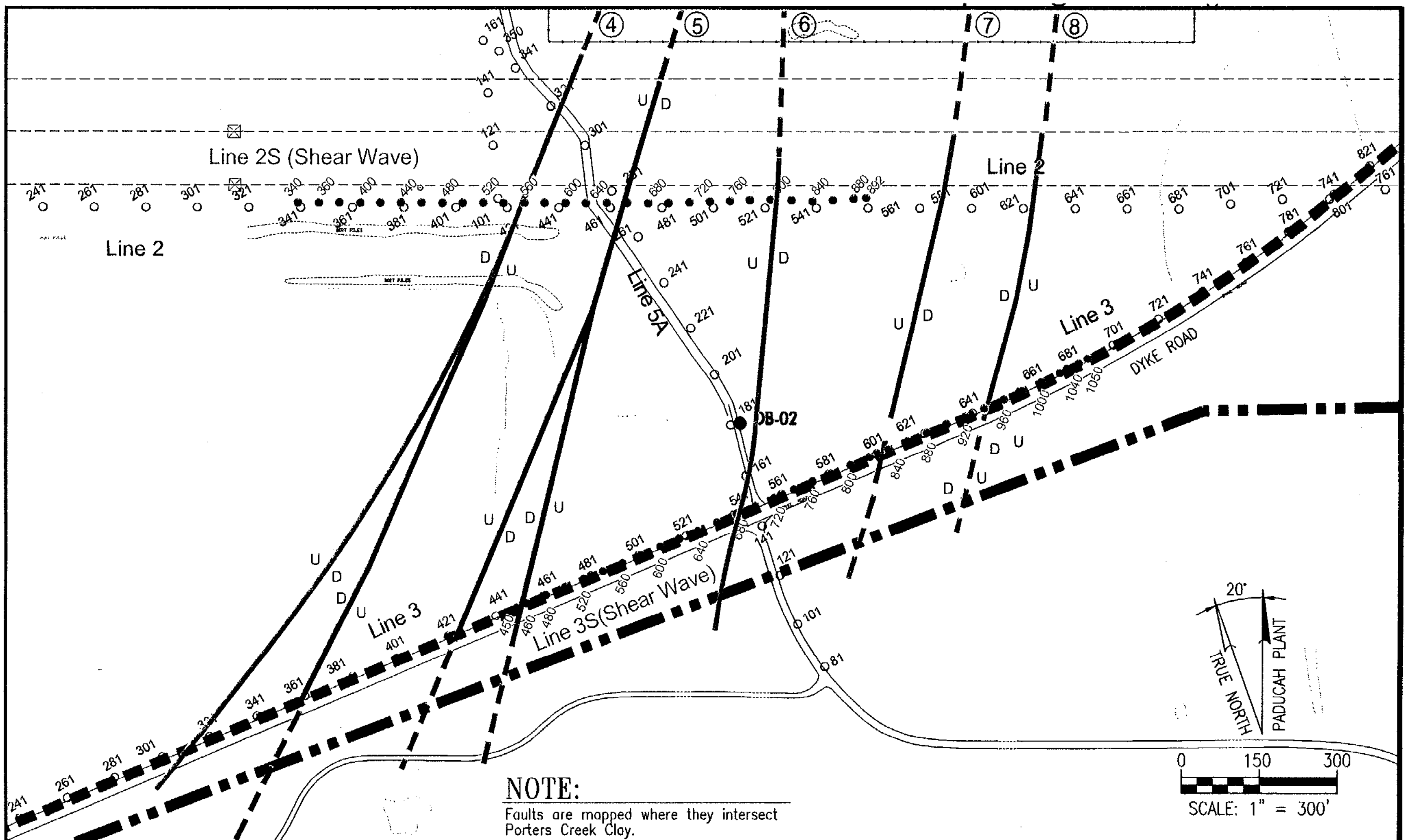
For reference, the interpreted faults from the p-wave survey sections are shadowed in white on the s-wave survey sections (Figs. 6.23 and 6.24). The faults interpreted from the p-wave survey generally occur in very close proximity to faults interpreted from the s-wave survey sections. Some small adjustments were made to positioning the faults because of the increased resolution provided in the s-wave survey data. The final interpreted faults are shown in red. Faults that are evident in the s-wave survey data, but were not seen on the p-wave survey sections, are shown in orange on Lines 2S and 3S.

Based on the s-wave survey profiles and associated DPT data, a total of five faults were investigated during the s-wave survey in the central portion of Site 3A. These profiles generally confirm the number and location of faults identified from the earlier p-wave survey. Figure 6.19 illustrated the faults interpreted from the p-wave survey data. Note that fault locations are mapped at the top of the limestone level. Figure 6.25 illustrates the spatial distribution of faulting after analysis of the s-wave survey sections. On this figure, the fault locations are mapped where they intersect the Porters Creek Clay reflector.

Overall, the s-wave survey sections confirm the faults interpreted from the p-wave data. Therefore, the same fault numbering system that was used in the p-wave survey has also been used for the s-wave survey. The s-wave survey data provide complementary information on Faults 4 through 8. A discussion of each of the potential faults is provided below.

- **Fault 3A:** Fault 4 was originally interpreted from the p-wave survey data to include a southwest-trending splay south of Line 2. With the additional resolution provided by the s-wave survey data, it now appears that there are two separate faults here (Fig. 6.25). The westernmost fault has been labeled "3A," as it was not previously identified as a separate entity. On Line 2S (Fig. 6.23), the fault indicators on the s-wave survey section are relatively weak; therefore, the fault plane is dashed. However, a sudden change in the deep reflector is evident, as well as a small potential offset in the firm sand reflector.

- **Fault 4:** This fault was interpreted from the p-wave survey sections to intersect the Porters Creek clay on Line 2S at approximately shot point 530 and bound the significant anticline-like feature along the west. However, the Porters Creek Clay reflector in this part of Line 2S is obscured by multiples, and data quality is diminished by poor surface conditions (Fig. 6.23). The s-wave survey interpreted fault is shifted slightly west from its original position, but the seismic indicators for the exact fault position are not obvious within this zone. Although confidence is high that there is a fault in the immediate vicinity of the location shown, Fault 4 is dashed to indicate the uncertainty in positioning. There are no significant anomalies in the seismic data to indicate that Fault 4 extends up to the firm sand.



LEGEND:

- ....Asphalt road
- ....Gravel road
- ....Pgdp boundary
- ....Site 3A boundary
- ....Fence line
- ....Stream
- ● DB-02..........PS suspension log
- ● 401..........S-wave survey location
- ○ 501..........P-wave survey location
- ....Interpreted fault location (Bedrock and unconsolidated dediments)
- ....Interpreted fault location (Bedrock and lower McNairy formation only)
- U/D ....Relative movement along fault plane

Fig. 6.25. Fault interpretation from p-wave survey.

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- **Fault 5:** Fault 5 was interpreted from the p-wave survey sections to be truncated beneath the McNairy unit (Fig. 6.19). The additional resolution provided in the s-wave survey data now contradicts this view. Fault 5 is interpreted to extend upward through the Porters Creek Clay and firm sand. Disrupted reflectors occur at the deep reflector and Porters Creek Clay levels, and velocity sags are interpreted at the firm sand level (Fig. 6.23). The s-wave expression of this fault is slightly west of its original, projected p-wave survey position (highlighted in white).

- **Fault 6:** Fault 6 is the only fault expected to be imaged on both Lines 2S and 3S (Fig. 6.25). The overall fault "signature" as it appears on both lines is quite similar. The fault plane is rotated slightly on Line 2S, and shifted slightly east on Line 3S, relative to the original p-wave interpretation. In addition, splay faults are evident on both s-wave survey sections above the deep reflector level (this was not apparent on the p-wave survey data). This fault is interpreted to be coincident with the eastern boundary of the significant anticline-like structure identified in the original p-wave survey data.

  On Line 2S (Fig. 6.23), the fault is defined by offset reflectors that are clearly evident at the deep reflector level. At the Porters Creek Clay, faulting is not as well defined, but there is an abrupt change in reflector character in the vicinity of where this fault should be. As discussed earlier, SCPT data indicate that variations in Porters Creek Clay sedimentation may be occurring here. At the firm sand and loess levels, localized dips in the reflector may be velocity sags, indicating fault-induced velocity variations.

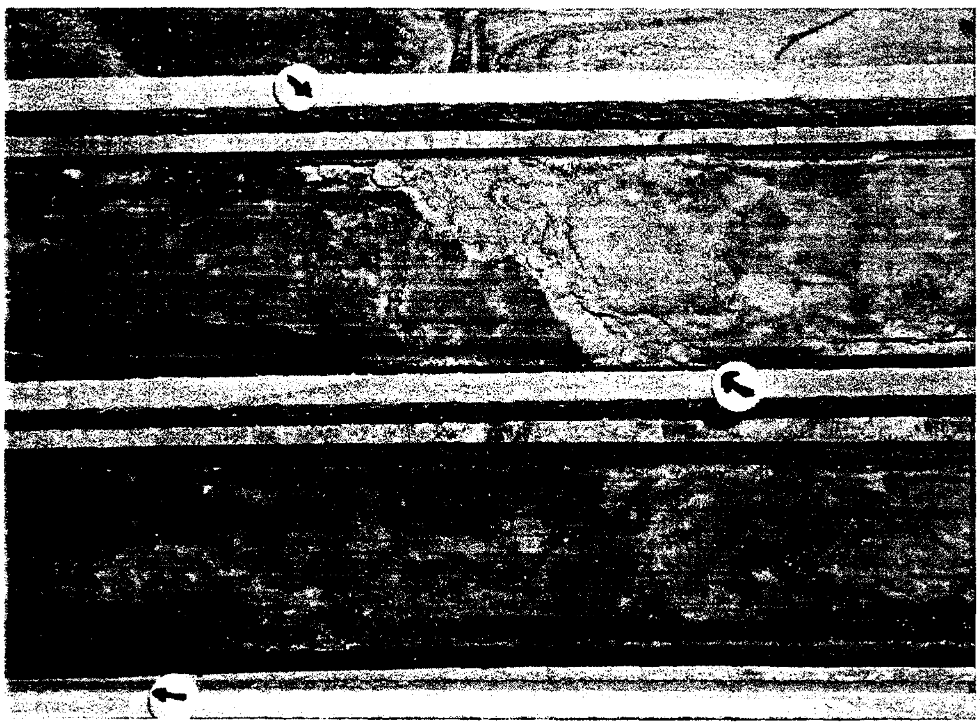
  Fault 6 is evident on Line 3S (Fig. 6.24) by offsets or otherwise disrupted reflectors at all levels. Nearby and to the west of the p-wave survey interpreted fault (highlighted in white), another fault and accompanying splay are newly interpreted. This new fault is interpreted to occur along the eastern flank of the anticline-like feature. Because this new fault is not evident on Line 2S, it appears that Fault 6 bifurcates into a series of narrow horsts and grabens as it trends south (Fig. 6.25).

  One DPT borehole was placed as close as possible to this fault in an attempt to confirm its presence. DPT L3-531 penetrated approximately 18 ft of loess. Several faults were intersected beginning a few feet below the base of the loess. Figure 6.26 is a photograph of this portion of the core. None of these features are the result of breakage because of sampling. The most obvious fault is the one at a depth of 28 ft that dips approximately 45° and separates the overlying silts and clays from the firm sand. In addition, two subtle steeply-dipping fault zones are located above this feature at depths between 22 and 26 ft bgs.

- **Fault 7:** Fault 7 occurs at roughly shot point 840 (Porters Creek Clay level) on Line 3S. Based on previous work, this fault is expected to occur east of Line 2S, and hence cannot be seen. The position of this fault is shifted slightly to the west from the original interpretation, and it is clearly evidenced on the s-wave survey section by offset reflectors at the deep reflector and Porters Creek Clay level. There is no indication that this fault extends upward to the firm sand.

- **Fault 8:** Fault 8 on the p-wave survey interpretation was imaged once again on Line 3S. This fault is the easternmost in the s-wave survey area, and based on the previous work, is not expected to be seen on Line 2S. On Fig. 6.25, the revised position of Fault 8 is slightly to the east and a splay fault is evident below the Porters Creek Clay level, extending east from the main fault plane. The fault is indicated by offset reflectors at the deep reflector and Porters Creek Clay levels, and by localized discontinuities in reflectors at the firm sand level. Above the firm sand, the interpretation becomes less certain, although small discontinuities at the loess level may indicate that faulting may extend into these sediments. Like most of the faults in this area, relative movement along the main fault plane is normal, with the downthrown side to the east. Sediments within the splay are downthrown and rotated relative to the sediments on either side.

(a). Location of fault plane in DPT L3-531 at Fault 6.



(b). Close-up of fault plane in DPT L3-531.

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Fig. 6.26. Fault plane observed in DPT L3-531.

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For most of the faults in this area, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks. The overall trend and geometry of the faulting is consistent with extensional regional tectonics and faulting observed in the Fluorspar Area fault complex of Massac County, Illinois, located just across the Ohio River.

Several of the faults identified in the initial p-wave survey data have been confirmed in the follow-up s-wave survey to extend upward into younger sediments overlying limestone bedrock, three of which are interpreted to extend to within approximately 20 ft of the ground surface.

6.3.3 Age Dating at Site 3A

The relative timing of the deposition of the sediments at Site 3A provides general age constraints. For example, the Porters Creek Clay is older than the overlying firm sand. The firm sand is older than the overlying loess. However, such relative age dating does not provide absolute ages of the deformation observed in the seismic reflection data or in the associated DPT boreholes. As such, all of the borings completed at Site 3A during the Seismic Investigation program were searched for pieces of detrital wood or charcoal for the purposes of ^{14}C dating. Only one sample identifiable as wood was recovered (CCGTSB0611). This very small twig was found in SB-06 at a depth of 11 ft. The remainder of the samples collected at Site 3A were from organic zones within soils.

A total of seven organic samples were collected. Their radiocarbon ages are presented in Table 6.4. The radiocarbon dates show that the loess is generally late Pleistocene in age with ^{14}C dates ranging from 13,540 to 15,620 years BP. The twig collected in SB-06 at a depth of 11 ft was dated to the early to mid-Holocene (6,830 years BP), suggesting that locally there are younger alluvial deposits that fill former erosional features in the loess. The uppermost soils (3 ft to 4 ft) yield dates of approximately 4000 years BP. These dates may represent the age of deposition or, more likely, a composite age of organic matter that has been incorporated to these depths due to bioturbation.

Table 6.4. Summary of organic samples and ^{14}C age dating[1] at Site 3A

Sample number	Boring	Depth (ft)	Measured ^{14}C age (years BP)[2]	Implication
CCGTD440L2	440-L2	10	13,540 ± 60	Age of loess
CCGTD500L2	500-L2	3.2	3,770 ± 50	Age of shallow soils
CCGTD620L3	620-L3	10	13,850 ± 60	Age of loess
CCGTD670L3	670-L3	10.2	15,620 ± 70	Age of loess
CCGTSB03C04	SB-03	4	4,190 ± 40	Age of shallow soils
CCGTSB03C34	SB-03	34	7,230 ± 40	Implication unclear – suspect sample fell from shallower depth during drilling
CCGTSB0611	SB-06	11	6,790 ± 40	Age of younger alluvium deposits

[1]Dates are reported as radiocarbon years before present (BP), where "present" is defined as 1950 A.D.
[2]Measured ^{14}C ages are based on the observed half life of ^{14}C.

One sample, CCGTSB03C34, yields questionable results. That sample was collected in soil boring SB-03 at a depth of approximately 34 ft. This level is well below the base of the loess and at or below the firm sand. Based on other ^{14}C age constraints, the age of 7230 years BP is much younger than would be expected, and totally inconsistent with the local stratigraphic section. It should be noted that this was the first SPT boring at Site 3A and was an uncased mud rotary hole. Because of problems with the safety hammer the upper portions of this hole were open for almost a week before the sample was collected. The sample was collected from near the top of a split spoon sampler, an area where sloughing from above can occur. Based on its anomalous age and its position in the sampler, it is likely that this sample originated nearer the surface and fell down the open hole. Consequently, this date should be viewed with this uncertainty in mind.

6.3.4 Results of the Site-Specific Fault Study

The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks.

These normal faults were formed when the bedrock was in tension, but today the bedrock is in compression. Therefore, the stresses that caused the faults to develop no longer exist. However, some of the faults have likely been reactivated as strike-slip faults under the current compressional stress regime.

Several of the faults identified in the p-wave survey and further studied during the s-wave survey extend through the Porters Creek Clay and into the materials underlying the surficial loess deposits. Three of these faults extend to within approximately 20 ft of the surface. A DPT borehole drilled adjacent to one of the postulated shallow faults encountered three fault planes at depths between 22 and 28 ft. No faults were observed in the overlying loess sampled in this same DPT borehole. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age with ^{14}C dates ranging from about 13,500 to 15,600 years BP.

Therefore, this study did not find Holocene-age displacement of faults at Site 3A.

7. SEISMIC DESIGN MODEL

As discussed in Chap. 1, the Project Core Team developed a list of seven questions that, when answered, would address seismic issues related to the siting of a potential CERCLA waste disposal facility at PGDP. This chapter develops a seismic design model of Site 3A, using the information presented in Chap. 3, to address Questions 6 and 7 posed by the Project Core Team. These two questions follow:

- Question 6. What is the peak ground acceleration (PGA) at the potential disposal facility site?
- Question 7. What are the characteristics of the design ground motion?

Previous ground motion studies at PGDP have been conducted for other locations on the DOE property. Site 3A is at a different location and the soil column from the top of rock to the ground surface contains a significant thickness of the Porters Creek Clay that does not exist at most of the other locations. Therefore, the main purpose of this study is to determine top-of-soil PGA and ground motion characteristics (ground shaking frequency, velocity, and displacements) pertaining directly to the Site 3A location.

Table 7.1 repeats the questions and presents a summary of the answers based on the seismic design model that was developed for Site 3A. Data, information, and details used to develop the responses to the questions presented above are included in the body of this chapter and other chapters of this document.

Table 7.1. Summary of answers to Questions 6 and 7 posed by the Project Core Team

Question	General answer
6. What is the PGA at the potential disposal facility site?	Based upon data collected from Site 3A, the PGA at Site 3A is calculated to be 0.48g for a 2500-year return period earthquake.
7. What are the characteristics of the design ground motion?	The design ground motions at Site 3A would be the same as those presented in (REI 1999). The shear-wave velocities in the soil column at Site 3A are similar to those measured at other locations on the DOE property, resulting in similar design ground motions.

PGA = Peak ground acceleration

7.1 BACKGROUND ON SEISMIC DESIGN

7.1.1 Seismic Regulations

EPA and the Kentucky Division of Waste Management (KDWM) regulations require that a municipal solid waste landfill located in a seismic impact zone be designed to resist the maximum horizontal acceleration anticipated for the site in lithified (rock) material. A seismic impact zone is an area with a 10% or greater probability that the maximum horizontal acceleration in lithified material expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 250 years (EPA 1995; KDWM 2002). Such an event would have a "return period" of 2500 years; that is, on average (over tens of thousands of years), the event would be exceeded once every 2500 years.

The maximum horizontal acceleration that actually occurs during an earthquake, and is felt by a structure on the ground surface, is referred to as the PGA of that earthquake. Seismic regulations, therefore, require that any landfill in a seismic impact zone be designed to resist its site-specific PGA having a 2500-year return period.

7.1.2 Probabilistic Seismic Hazard Assessment

The maximum horizontal acceleration in rock is depicted on USGS seismic hazard maps (Frankel et al. 1996), or may be developed based on a site-specific seismic risk assessment. If the PGA is larger than 0.10 g, then the site is considered to be in a seismic impact zone and the disposal cell must be designed for earthquake loads at the PGA value. Based on the USGS seismic hazard maps, the top-of-rock PGA for Site 3A would be 0.8 g; therefore, Site 3A is located in a seismic impact zone.

A site-specific study is performed using a probabilistic seismic hazard analysis (PSHA) to determine more precisely the maximum horizontal acceleration at a particular site. The PSHA requires an assessment of past earthquakes in the region. Based on the sizes and locations of these past earthquakes and other geologic and geophysical data, the PSHA determines the seismic hazard at the study site. The seismic hazard is displayed as a plot of annual probability of exceedance (inverse of the return period) versus a range of PGAs, as shown in Fig. 7.1. The PGA for a 2500-year return period (annual probability of exceedance of 4E-04) can then be determined from the curve.

When conducting a PSHA to determine the seismic hazard of a site, the seismic risk in terms of the probability of a PGA value being exceeded during a given time can also be broken down into what types of earthquakes are causing most of the risk: "Are they small to moderate earthquakes near the location of interest?" or "Are they large earthquakes at great distance from the site?" This is known as deaggregating the hazard. Understanding the deaggreation of the earthquake risk provides information that is helpful to understanding soil amplification issues (discussed in Sect. 7.1.3) and the potential frequency of ground shaking (discussed in Sect. 7.1.4).

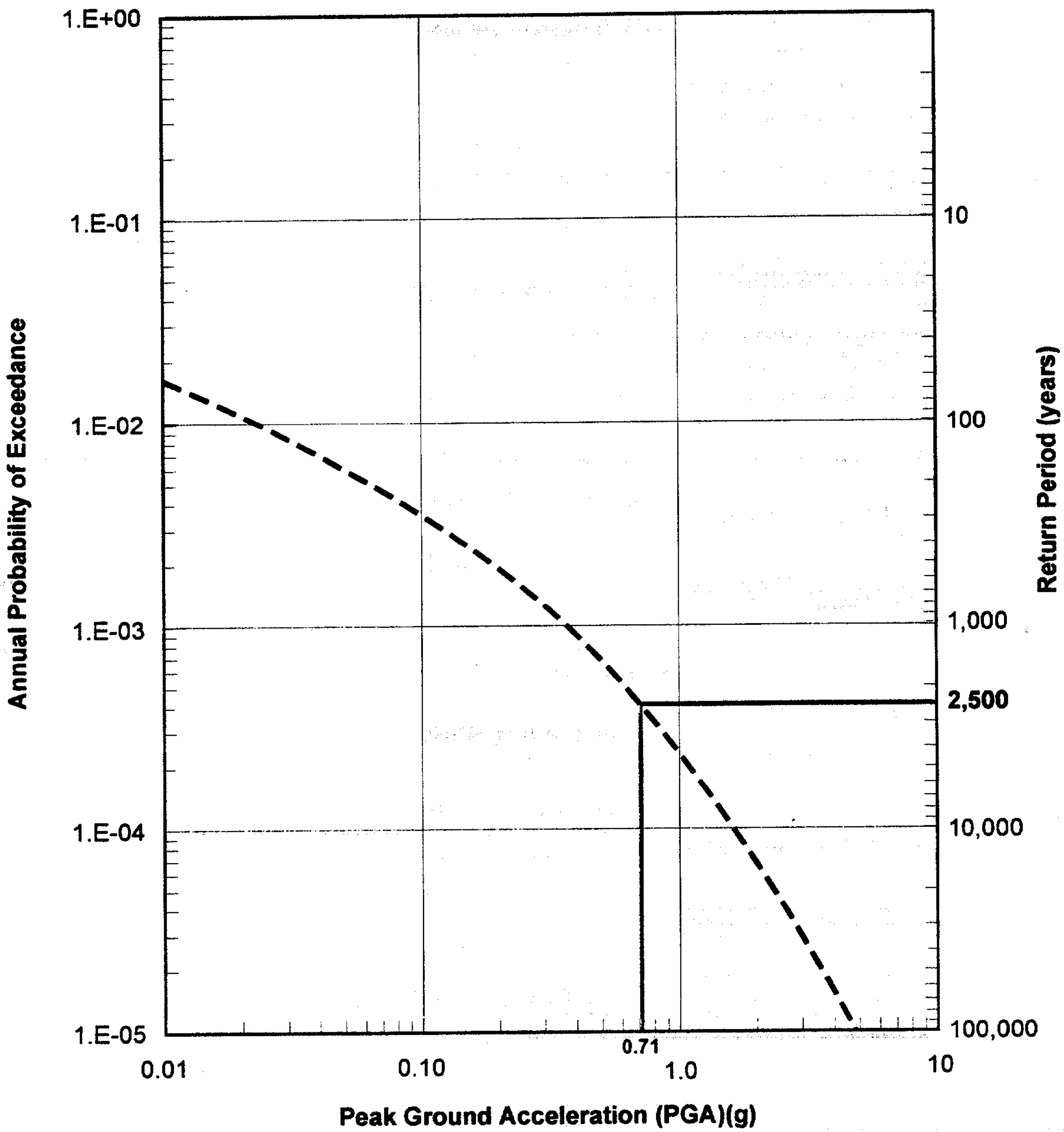
7.1.3 Soil Amplification

The PGA is typically computed at the top of bedrock, which at Site 3A is between 325 and 425 ft bgs (Fig. 3.9). However, any potential waste disposal facility would be founded on soil materials up to 425 ft thick overlying bedrock. As a result, the PGA at the top of the soil (at the base of the disposal cell) must be determined.

Soils typically amplify the rock PGA. Soil amplification factors can vary considerably, depending on the type of soil, depth of soil above the rock, top-of-rock PGA, and ground motion frequency. If the soil materials remain linear (Sect. 7.1.4) and the top-of-rock PGA is small, then the soil amplification factor can be as high as 4.0. However, if the soil materials become nonlinear, as happens with a very high top-of-rock PGA, then the soil amplification factors can be as low as 0.5. Soil amplification calculations were completed for Site 3A, as discussed in Sect. 7.3.

7.1.4 Structural Amplification and Damping

Structures, including earthen structures like a disposal cell, respond to earthquakes in different ways depending on frequency characteristics, damping, and material performance. Frequency characteristics determine how a landfill will shake during an earthquake. If the fundamental frequency of the disposal cell is similar to the earthquake's dominant frequency, it can experience significant amounts of acceleration and displacement at the top of the landfill. Whereas the top-of-soil PGA is the maximum acceleration that is being felt by the foundation of the structure, the acceleration at the top of the structure could be two to three times greater than that PGA. This amplified acceleration at the top of the structure is known as the peak spectral acceleration (PSA).



Source data: REI(1999)

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Fig. 7.1. Seismic hazard curve for PGDP.

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To depict the frequency characteristics of the structure, a plot of the PSA versus ground motion frequency is developed. This plot, known as a uniform hazard spectra (UHS), provides the PSA for each frequency from 0.1 Hz (10 sec.) to 100 Hz (0.01 sec.). The shape of the UHS curve represents the relationship between the earthquake acceleration and the ground motion frequency, which can be converted into ground motion velocities and displacements. The UHS is uniform in that the probability of exceeding the PSA at any frequency is the same as exceeding the PGA. UHS frequencies of 0.1 to 1 Hz are known as long period motions and the PSA values are most often much less than the PGA value. UHS frequencies of 5 to 10 Hz are known as short period motions and the PSA values are always larger than the PGA value. At a structural frequency of 100 Hz, all structures relative to earthquake ground motion frequencies are rigid and no amplification in the structure occurs. As a result, at a frequency of 100 Hz the PSA and PGA are the same.

While structural amplification can increase the acceleration, damping absorbs the earthquake energy and de-amplifies the acceleration, reducing the PSA values. Damping values for structures that do not exceed structural material elastic (linear) limits typically range from 2 to 10%. As a result, UHS plots are often developed for structures having 2, 5, 7, and 10% damping (REI 1999). The larger the damping value, the lower the PSA values.

Material performance of structures and soils during earthquakes can be considered as having either of two states, linear or nonlinear. The UHS represents the performance of a structure in a linear state for design purposes. Design, or evaluation, of a structure in the nonlinear state is performed using simplified equivalent linear analysis techniques or computationally complex nonlinear analysis. In the nonlinear state, a structure absorbs the earthquake energy and the PSA values can become smaller than the PGA values, as discussed below.

7.1.5 Methods for Determining Soil Amplification

There are two methods for determining soil amplification that are considered state-of-practice regulatory status in building codes and NRC guidance. Method 1 involves using a PSHA to develop top-of-rock spectra representing the baseline seismic hazard. An equivalent linear method is used to determine site-specific soil amplification factors. These factors are then applied to the top-of-rock spectra to determine top-of-soil spectra.

Method 2 is similar to Method 1 but follows NRC Regulatory Guide 1.165 (NRC 1997). Two "controlling earthquakes" are selected and used to develop site-specific top-of-rock response spectra. A site-specific soil amplification analysis is then performed considering uncertainties in site-specific geotechnical properties to determine top-of-soil response spectra. Depending on details of the deaggregation parameters, Methods 1 and 2 may yield the same results.

More recently, three alternative methods have been introduced for determining site-specific soil amplification. These methods, referred to as "fully probabilistic," go beyond current state-of-practice and are much more computationally intensive. They have been introduced because some experts believe that Methods 1 and 2 may underestimate the soil amplification and seismic hazard in some cases.

Method 3 (McGuire et al. 2001) incorporates the soil amplification factors and factor uncertainty directly into the PSHA integration. It is known that this method will give higher values for a given seismic source model than either Method 1 or 2 because it incorporates values above the median site amplification factors into the probability-of-exceedance calculation. The amount of this increase depends on how the characterization of uncertainty in soil column amplification factors is incorporated. In Method 3, this uncertainty is added on top of all other model uncertainties in a way that maximizes the results.

Method 4 is an alternative way to incorporate site-specific soil amplification factors and their uncertainties. This method has been used at other DOE sites (Lee et al. 1998). In this approach, the "hard rock" PSHA is used together with the magnitude (a measure of the strength of the earthquake generally as determined from seismographic observations) and ground-motion-level-dependent distributions on site amplification to develop a soil surface PSHA. The soil surface PSHA is computed without modification or adjustment to the original "hard rock" PSHA. Like Method 3, this method is expected to provide results higher than Methods 1 and 2.

Method 5 (McGuire et al. 2001) is an alternative that replaces the use of "hard rock" attenuation relationships for the probabilistic seismic hazard analysis (e.g., Methods 1 and 2) with site-specific soil attenuation relationships that directly incorporate model uncertainties, including soil column uncertainties, into the UHS calculation. This method handles model uncertainties more realistically.

7.2 PREVIOUS SEISMIC HAZARD STUDIES AT PGDP

Numerous seismic hazard studies have been conducted at the PGDP over the past 25 years. REI completed the most recent study using a PSHA and Method 1 soil amplification methodology (REI 1999). Top-of-soil UHS plots were developed for return periods of 250, 500, 1000, and 5000 years. The effect of the soil column was incorporated by scaling the probabilistic hard-rock UHS values using median values of site-specific soil amplification factors (REI 1993). The soil column was based on measured shear-wave velocity profiles taken from four deep borehole clusters drilled on the DOE property (Staub and Wang 1991). Fig. 7.2 shows the locations of the four deep borehole clusters plus the location of the deep borehole cluster (DB-01 and DB-02) drilled at Site 3A during this Seismic Investigation.

In 2001, a study was conducted to develop ground motion values for an earthquake having a return period of 2500 years (Beavers 2001). Results of the 1999 REI study were interpolated (using a linear interpolation) for a return period of 2500 years, resulting in a PGA of about 0.8 g at bedrock and 0.5 g at the top of soil. Ground motions at the top of soil were presented as spectral shapes for various damping values. USGS seismic hazard maps (Frankel et al. 1996) showing top-of-rock PGA for a firm rock site were interpolated to obtain the top-of-rock PGA for a hard rock site (such as PGDP) of approximately 0.8 g. Because both methods resulted in the same top-of-rock PGA for PGDP, it was concluded the methods were compatible. Therefore, although USGS maps do not directly provide top-of-soil ground motions, it could be concluded that a top-of-soil PGA of 0.5 g would be appropriate for design of a potential on-site CERCLA waste disposal facility.

USGS (Cramer 2001) used a PSHA and Method 3 soil amplification methodology to determine top-of-soil PGA at PGDP. This resulted in a higher PGA of 0.7 g. KDWM expressed concerns about this value but agreed to a site-specific evaluation of the ground motion values (KDWM 2002).

As a result, a reassessment of the PGA at PGDP was conducted (BJC 2002c). A log-log interpolation of the REI 1999 data, which is considered a more accurate interpolation, was used to determine top-of-rock PGA for the 2500-year return period earthquake. This resulted in a top-of-rock PGA of 0.71 g. The study concluded there was satisfactory agreement between the REI top-of-rock PGA hazard curve and that generated by BJC. Based on the REI 1999 soil amplification value of 0.67, the resulting top-of-soil PGA at PGDP would then be 0.48 g.

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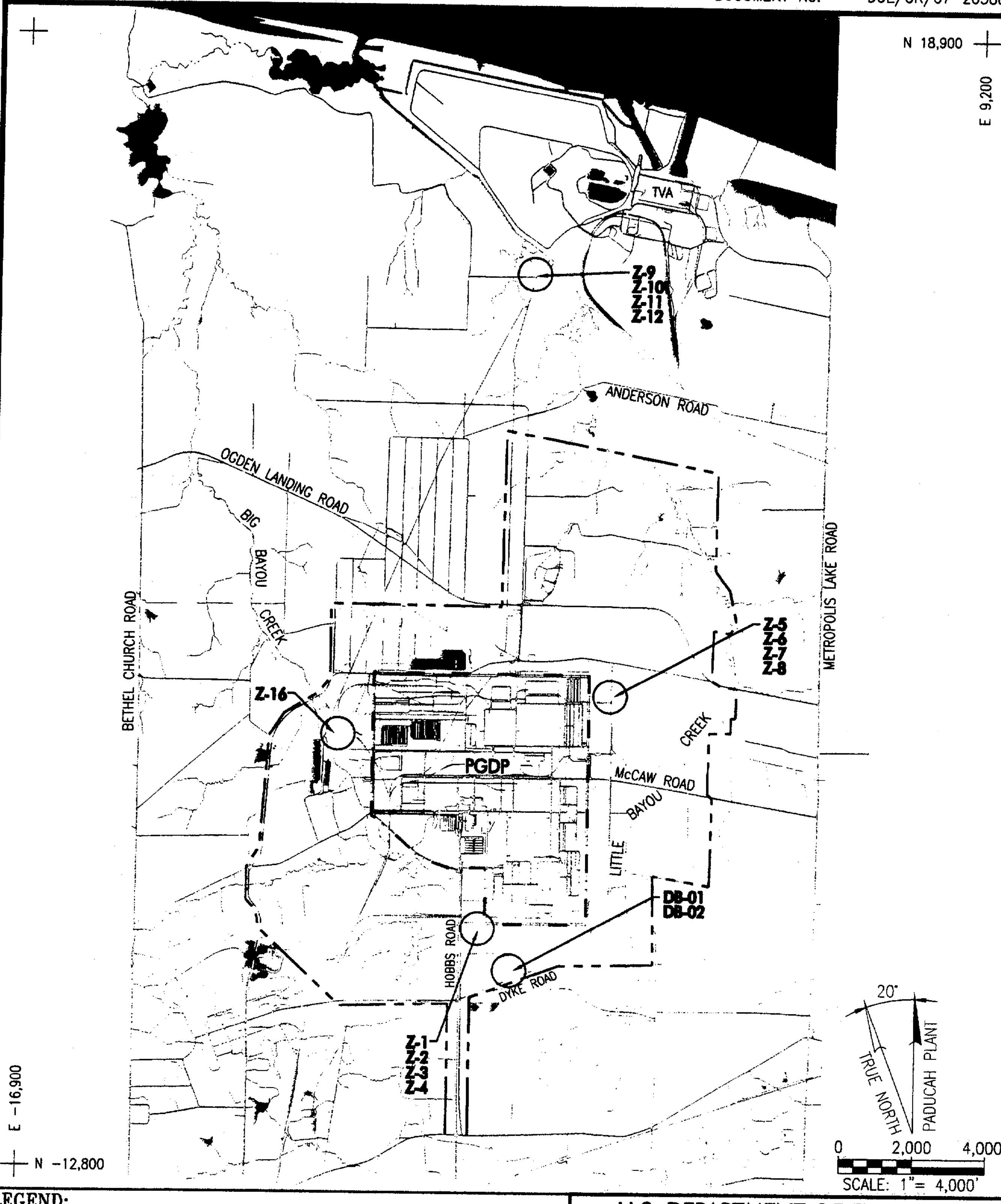

LEGEND:

.......DOE property boundary
...............PGDP boundary
.................Asphalt road
.... Stream

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Fig. 7.2. Location of deep boreholes in the PGDP vicinity.

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The BJC 2002 study developed soil-site amplification factors using all five soil amplification methods, including two variations, for a total of seven. The top-of-soil PGA value ranged from 0.48 g to 0.57 g with values of 0.48, 0.48, 0.57, 0.56, 0.47, 0.48, and 0.52 g with a mean value of 0.51 g. The study concluded that, given the uncertainties associated with seismic parameters at the PGDP site, the REI (1999) ground motions should be used for design at PGDP (i.e., the 0.48 g value).

7.3 SITE 3A GROUND MOTIONS

Based on the BJC 2002c study, it is concluded that a top-of-rock PGA of 0.71 g is appropriate for a 2500-year return period earthquake at PGDP. The 2500-year return period UHS rock motion shown in Fig. 7.3 (as developed by BJC 2002c from the REI 1999 study) was used as the top-of-rock spectra to determine new top-of-soil ground motion spectra for a potential CERCLA waste disposal facility at Site 3A.

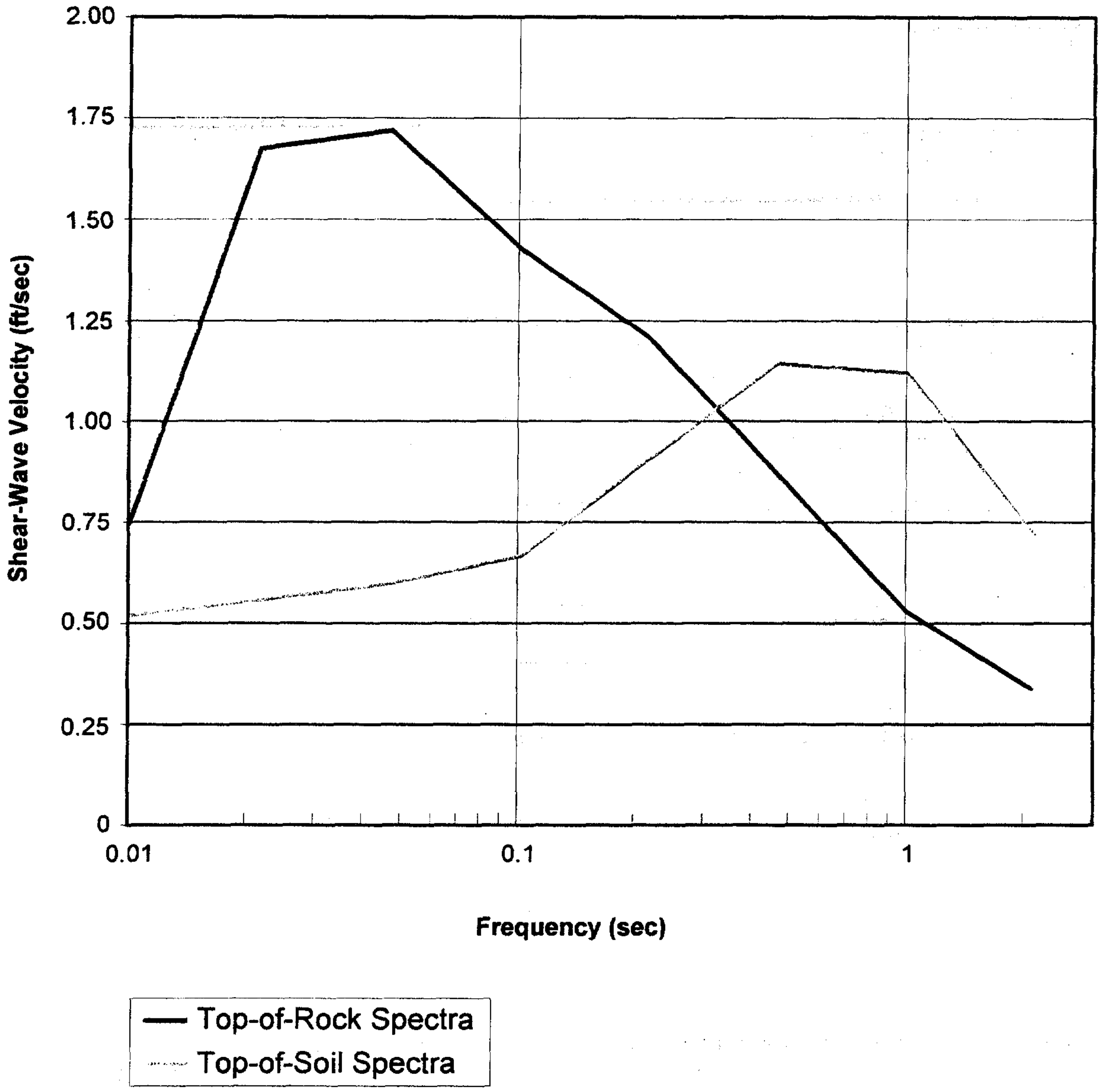
Based on the BJC 2002c study, it was also concluded that Method 1 provides a good representation of soil amplification. As a result, Method 1 was used to determine Site 3A ground motions using the shear-wave velocities measured at Site 3A (Sect. 3.7). Shear-wave velocities were measured in a deep borehole (DB-02) at 1.64-ft (0.5-m) intervals to a depth of about 374 ft and in SCPT soundings throughout Site 3A that were probed to depths of up to 60 ft bgs. For the soil column down to a depth of about 60 ft, all data collected from the SCPT soundings and borehole DB-02 were averaged. For the remaining depth to the top of bedrock, the shear-wave velocity data collected from Borehole DB-02 was used. The resulting shear-wave profile for the soil column at Site 3A is shown in Table 7.2. This shear-wave profile was then used to determine the soil amplifications for Site 3A using Method 1.

Table 7.2. Site 3A shear-wave velocities

Elevation bottom of soil zone (ft msl)	Shear-wave velocity, Vs (ft/sec)	Standard Deviation (ft/sec)
380	607	±162
368	921	±208
360	1250	±352
348	913	±159
340	1068	±251
300	1028	±202
242	1141	±69
217	1430	±179
140	1532	±213
-10	1700	±154

Figure 7.4 shows a comparison between Site 3A shear-wave velocity profile and the shear-wave velocity profile used by REI (1999). There is little difference between the two sets of data, except for the deeper portion of the profile where the variation is on the order of 20%. A variation of less than 20% is not expected to change the soil amplification factors to any extent.

As stated above, Method 1 uses an equivalent-linear method to determine soil amplification factors that are applied to the top-of-rock spectra to determine top-of-soil spectra. To determine soil amplification, a random vibration theory analysis is used. This analysis has shown to be a robust method of incorporating uncertainty and randomness of dynamic material properties into the computed response. The analysis randomly varies the shear-wave velocity profile and the thickness of the material layers. The



Note: Spectra based on 2500-year return period and 5% damping.

Fig. 7.3. Uniform Hazard Spectra (UHS) for ground motions at Site 3A.

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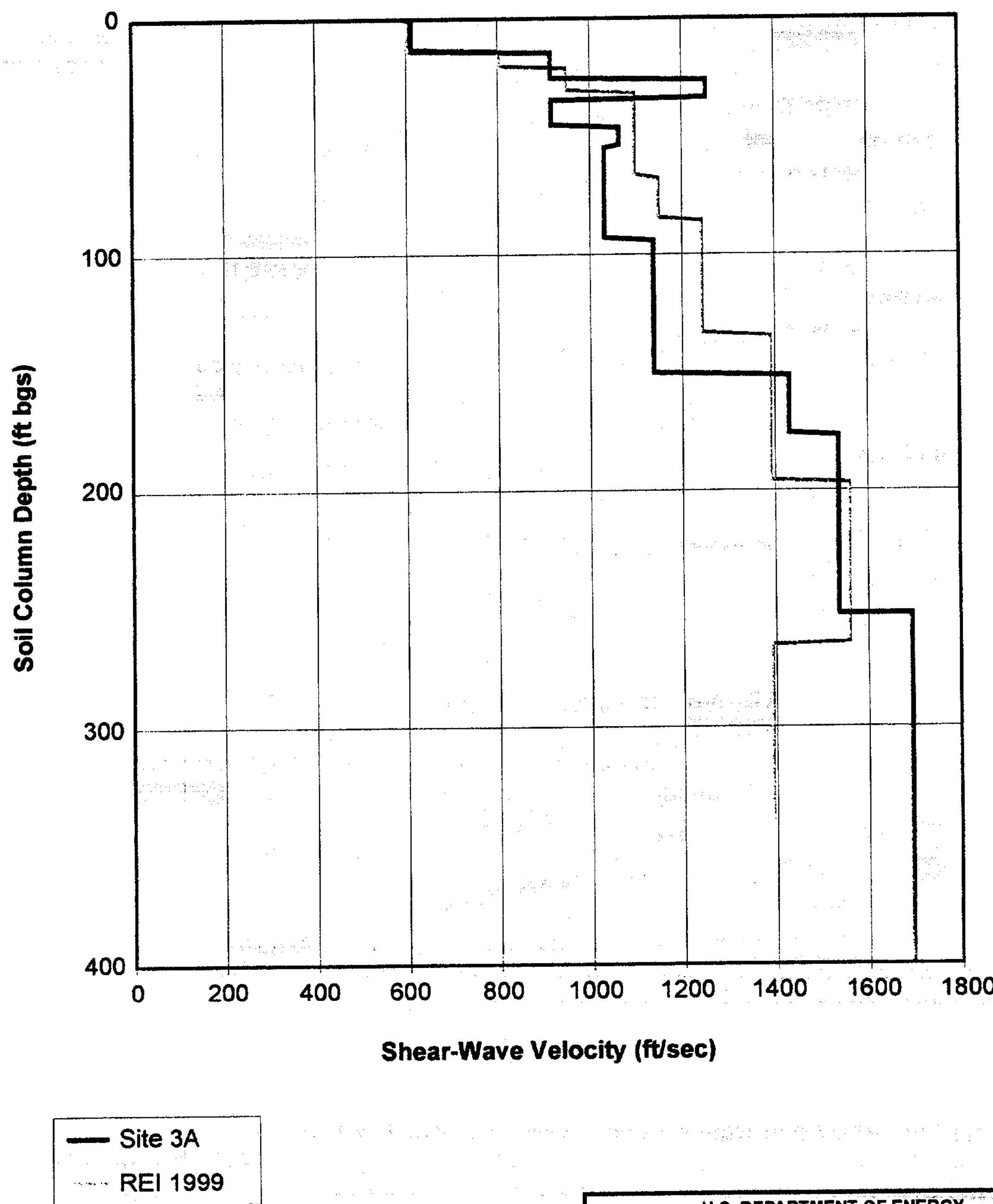

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Fig. 7.4. Comparison of shear-wave velocity profiles (Site 3A vs. REI 1999).

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randomization is based on a correlation model developed from a statistical analysis on about 500 measured shear-wave velocity profiles (EPRI 1993; Silva et al. 1997). Profile depth (depth to bedrock) is also varied on a site-specific basis using a uniform distribution. Variation of the profile depth is important at Site 3A because, as noted in Chap. 3, the top-of-rock elevation varies from 50 ft above msl to 40 ft below msl.

To accommodate uncertainty in other geotechnical materials of the soil column on a generic basis, the data are independently randomized about the base case values. This randomization is based on statistical analysis of laboratory test results.

As done in the REI 1999 study, to accommodate nonlinear soil response, the generic cohesionless soil G/G_{max} and hysteretic damping curves developed by EPRI (1993) were used. These curves accommodate the effects of confining pressure and have been developed based on laboratory tests as well as a careful literature review.

Soil amplification calculations were performed for ground-motion amplitudes associated with top-of-rock PGA values of 0.05, 0.10, 0.20, 0.30, 0.40, 0.50, 0.75, and 1.00 g. Appendix F shows the calculated amplification factors as a function of frequency for the various ground-motion amplitudes. These amplification factors were then used to convert the spectral accelerations at top-of-rock to spectral accelerations at top-of-soil for Site 3A. The 5% damped spectral shape shown in Fig. 7.3 was used. Results of these calculations determined that, for a top-of-rock PGA of 0.71 g, the soil amplification is 0.67 (Appendix F). This results in a top-of-soil PGA of 0.48 g for a 2500-year return period earthquake at Site 3A.

This value is equal to the top-of-soil PGA value of 0.48 g calculated by REI in its previous study at PGDP. Therefore, the recommended top-of-soil PGA for the design of a potential CERCLA waste disposal facility at Site 3A is 0.48 g.

Because the shear velocities in the soil column at Site 3A are similar to those determined previously at other locations on the DOE property, similar design ground motions would be expected to those previously calculated for PGDP. The design ground motions at Site 3A would be the same as those presented in REI 1999. A uniform hazard spectra that relates ground acceleration to the ground shaking frequency is presented in Fig. 7.3 to define the design ground motions at Site 3A.

7.4 SUMMARY OF THE SEISMIC DESIGN MODEL

A PSHA was performed to determine the PGA and other related ground motions (ground shaking frequency, velocity, and displacements) for an earthquake having a 2500-year return period. The corresponding PGA value at the top of rock (400 ft bgs) was determined to be 0.71 g.

Because the potential CERCLA waste disposal facility would be founded on soil materials up to 400 ft thick, further analysis was done to calculate the PGA at the top of the soil (at the base of the disposal cell). Site-specific soil amplification factors were calculated for Site 3A based on the shear-wave velocities measured in the deep borehole (DB-02) and SCPT soundings. The soil amplification factor for a top-of-rock PGA of 0.71 g was calculated to be 0.67. This results in a top-of-soil PGA of 0.48 g at Site 3A for a 2500-year return period earthquake.

This value is equal to the PGA value of 0.48 g interpolated from REI by BJC (2002c) in its previous re-evaluation study at PGDP. Therefore, the recommended top-of-soil PGA for the design of a potential CERCLA waste disposal facility at Site 3A is 0.48 g.

The shear-wave velocities in the soil column at Site 3A are similar to those determined previously at other locations on the DOE property, resulting in similar design ground motions. Therefore, the design ground motions at Site 3A would be the same as those determined by REI. A uniform hazard spectra that relates ground acceleration to the ground shaking frequency is presented in Fig. 7.3 to define the design ground motions at Site 3A.

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8. SUMMARY AND CONCLUSIONS

This Seismic Investigation report has been prepared to summarize and present conclusions from a regional and site-specific Seismic Investigation at PGDP. The field investigation was developed to address seismic issues associated with potentially siting a CERCLA waste disposal facility at PGDP, in particular at Site 3A. The field investigation consisted of a Paleoliquefaction Study, regional Fault Study, site-specific Fault Study, and Geotechnical Study. The results of these studies have been presented in this report.

The Project Core Team developed the following seven questions that, when answered, would fully address the seismic issues:

1. Is there evidence of paleoliquefaction?
2. Is there paleoseismic evidence of local strong motion?
3. Is there potential for future liquefaction?
4. Is there evidence of Holocene displacement of faults at PGDP?
5. Are there faults underlying the potential disposal facility site?
6. What is the peak ground acceleration (PGA) at the potential disposal facility site?
7. What are the characteristics of the design ground motion?

Table 8.1 repeats these questions and presents a summary of the answers developed during the Seismic Investigation. The following sections summarize the conclusions of each study.

8.1 PALEOLIQUEFACTION STUDY

The Paleoliquefaction Study was developed to address Questions 1 and 2, and to support answering Questions 3, 6, and 7. The study included a review of historical information on liquefaction in the region, a search for evidence of paleoliquefaction features in the region, an evaluation of borehole cores taken from Site 3A for evidence of past liquefaction, and an evaluation of the results of laboratory testing of soil samples collected from Site 3A to assess liquefaction potential. Paleoliquefaction is defined here as seismically-induced liquefaction features associated with prehistoric Holocene or late Pleistocene earthquakes.

Field investigations conducted as part of the Seismic Investigation found no large liquefaction features along the Ohio River in the vicinity of PGDP. The riverbank afforded adequate exposure of the sediments such that if large liquefaction features were present they should have been obvious. Smaller-scale paleoliquefaction features may have been present but were not observed due to their relatively small size or the typical veneer of river deposits and vegetative cover.

Field investigations conducted along portions of Bayou and Little Bayou Creeks found no definitive evidence of paleoliquefaction at the PGDP.

The literature does report some small liquefaction features within 15 miles of the PGDP. The closest are located along the banks of the Ohio River, about 8 miles to the northeast. These features are in the general vicinity of Fort Massac, Illinois, a location where liquefaction was reported during the February 7, 1812, New Madrid earthquake. These features were small and relatively unweathered, suggesting that they were probably outlying liquefaction features resulting from the 1811 and 1812 New Madrid earthquakes. Small liquefaction features are also reported in the literature along the Post Creek Cutoff, about 12 miles northwest of PGDP.

Table 8.1. General answers to Project Core Team questions to address seismic issues at Site 3A

Question	General answer
1. Is there evidence of paleoliquefaction at or near PGDP?	Field observations made along the Ohio River in the vicinity of PGDP found no large liquefaction features. Smaller scale paleoliquefaction features may have been present but remained unobserved because of their relatively small size or veneer of river deposits and vegetative cover. There is no definitive evidence of paleoliquefaction at PGDP based on results of field investigations conducted along portions of Bayou and Little Bayou Creeks. The literature does report some small liquefaction features located along the banks of the Ohio River, about 8 miles northeast of PGDP, and along the Post Creek Cutoff, about 12 miles northwest of PGDP.
2. Is there paleoseismic evidence of local strong ground motion?	The absence of large paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. The small liquefaction features that have been reported in the literature are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.
3. Is there potential for future liquefaction at Site 3A?	Many of the soils present at the site are clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet the criteria that distinguish those fine-grained soils that could experience large-scale strain, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. Some liquefaction within the sands and deformation within the silts and clays could occur at PGAs approaching 0.5 g.
4. Is there evidence of Holocene displacement of faults at PGDP?	This study did not find Holocene displacement of faults at Site 3A. Several faults identified in seismic reflection data at Site 3A have been confirmed to extend through the Porters Creek Clay and into the materials underlying the surficial loess deposits. Three of these faults are interpreted to extend to within approximately 20 ft of the ground surface. One DPT borehole encountered three fault planes at depths between 22 ft and 28 ft. No faults were observed in the overlying loess. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age with ^{14}C dates ranging from 13,500 to 15,600 years BP. At the Barnes Creek site located 11 miles northeast of PGDP, this study found Holocene age displacement of faults in deposits with ^{14}C dates ranging from 5000 to 7000 years BP.
5. Are there faults underlying the potential disposal facility site?	The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults beneath Site 3A, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks. Several of the faults extend through the Porters Creek Clay and into the materials underlying the surficial loess. Three of these faults extend to within 20 ft of the ground surface.
6. What is the PGA at the potential disposal facility site?	Based upon data collected from Site 3A, the PGA at Site 3A is calculated 0.48 g for a 2,500-year return period earthquake.
7. What are the characteristics of the design ground motion?	The design ground motions at Site 3A would be the same as those presented in REI 1999. The shear-wave velocities in the soil column at Site 3A are similar to those determined previously at other locations on the DOE property, resulting in similar design ground motions.

BP = years before present, where "present" is defined as 1950 A.D.
DPT = direct push technology
PGA = peak ground acceleration
PGDP = Paducah Gaseous Diffusion Plant
REI = Risk Engineering, Inc.

The absence of large paleoliquefaction features within 15 miles of PGDP suggests that local strong ground motion has not occurred within the past few thousand years. In this context "local strong ground motion" is defined as strong ground motion resulting from a local earthquake. The small liquefaction features that have been reported in the literature are located in sediments that are especially prone to liquefaction and are probably associated with large earthquakes originating outside the area. It should be stressed that the available exposures may only provide a record for the late Holocene.

Many of the soils present at Site 3A are fine-grained clays and silts that by their very composition are not prone to liquefaction. In addition, laboratory evaluation of these materials found that they do not meet criteria that distinguish those fine-grained soils that could experience large-scale strain, similar to liquefaction. The sands encountered at Site 3A are generally firm and are not expected to liquefy under low to moderate levels of ground motion. However, based on the calculations presented in this report, it was concluded that some liquefaction within the sands and deformation within the silts and clays could occur at PGAs approaching 0.5 g.

8.2 FAULT STUDY

The purpose of the Fault Study is to determine whether Holocene-age faulting has occurred in the PGDP vicinity. The Fault Study is to answer Questions 4 and 5 posed by the Project Core Team and to assist in any subsequent facility design activities. The Fault Study included both regional and site-specific components.

8.2.1 Regional Fault Study

The regional Fault Study was conducted at the Barnes Creek site (Massac County, Illinois) to collect data to support the design of a potential on-site CERCLA waste disposal facility. Such data include displacement, earthquake magnitude, recurrence interval, and age of the most recent event. The study included two activities: (1) mapping of Barnes Creek and (2) GPR and DPT investigations of a nearby terrace graben area. These activities were implemented to identify the key geologic units, their relationship with observed faults, and their dates of deposition to establish ages of past fault movements. Although excavation of test pits and a trench was originally planned to collect visible evidence of shallow faulting, data collected from the creek banks and DPT boreholes were sufficient in dating the deposits and in determining that no correlation exists between the topography and faulting. Therefore, the test pits and trench were not excavated.

Geologic structures observed in Barnes Creek include individual joints, faults, clay dikes, and paired faults forming down-dropped blocks known as grabens. Neotectonic studies were carried out in a portion of Barnes Creek to determine if mapped faults have moved within the Holocene Epoch. Investigations in the creek identified five geologic units. The three oldest units, the Cretaceous McNairy Formation and the gravels, sands, and silts of both the upper and lower Metropolis Formation exhibit faults, clay dikes, and joints. The two youngest units, a surficial light brown sandy alluvium and an underlying light gray alluvium, did not exhibit faulting.

The trends (generally northeast-southwest) of the geologic structures in the oldest units and style of deformation are consistent with bedrock faults mapped to the north of the study area by the ISGS. The northeast-southwest trends are also consistent with the trend of the NMSZ to the southwest, suggesting that these features may be related.

The relative timing of the observed deformations in the geologic structures varies. A number of geologic structures are limited to the McNairy Formation and clearly pre-date deposition of the Metropolis materials.

Other features involve both the McNairy and Metropolis materials to the same extent, while others appear to be re-activation of old features in the McNairy after or during deposition of the Metropolis materials.

Radiocarbon ages confirm that repeated deformation has occurred at various times along some of the observed faults. Deformation began prior to the deposition of the lower Metropolis (late Pleistocene), continued during the deposition of the upper Metropolis (which is 5000 to 7000 years old), and most recently occurred in the mid-Holocene, after the deposition of the upper Metropolis (within the last 5000 years). Therefore, faults observed at the Barnes Creek site did extend into Holocene-age deposits. The maximum displacement observed in a single event is approximately 1 ft in the lower Metropolis.

Investigation of the terrace graben area concluded that the observed stratigraphy is consistent with a combination of two models: (1) a graben with up to 50 ft of displacement within the past 12,000 years, and (2) an erosional feature with up to 50 ft of infilling within the past 12,000 years. Radiocarbon ages in the terrace graben area indicate that the deep fine-grained sediments beneath the Metropolis are approximately 11,000 years old, indicating that the overlying Metropolis dates from the late Pleistocene or early Holocene.

8.2.2 Site-Specific Fault Study

The site-specific Fault Study at Site 3A was developed to answer Questions 4 and 5. The study included an initial p-wave survey followed by an s-wave survey and DPT boreholes. A GPR calibration survey found that GPR was not a viable investigative tool at Site 3A. Although excavation of test pits and a trench was planned so as to collect visible evidence of shallow faulting, field conditions were not amenable to excavation and, therefore, the test pits and trench were not excavated.

The site-specific Fault Study identified a series of faults beneath Site 3A. For most of the faults, relative movement along the main fault plane is normal, with the downthrown side to the east. These normal faults, along with their associated splays, either form a series of narrow horst and graben features, or divide the local sediments into a series of rotated blocks.

Several of the faults identified in the p-wave survey extend through the Porters Creek Clay at an approximate depth of 30 to 60 ft bgs and into the materials underlying the surficial loess deposits. Three of these faults extend to within approximately 20 ft of the ground surface. A DPT borehole drilled adjacent to one of the postulated shallow faults encountered three fault planes at depths between 22 and 28 ft. No faults were observed in the overlying loess sampled in this same DPT borehole. The radiocarbon dating at Site 3A found that the loess is late Pleistocene in age, with ^{14}C dates ranging from about 13,500 to 15,600 years BP.

Therefore, there is no direct evidence of Holocene displacement of faults at Site 3A. However, faults observed at the Barnes Creek site did extend into Holocene-age deposits.

8.3 GEOTECHNICAL STUDY

The Geotechnical Study was developed to acquire seismic and geotechnical characteristics of the deposits at Site 3A for use in the design of a potential on-site CERCLA waste disposal facility. These site-specific soil properties were used in answering Questions 3, 6, and 7. The study included drilling, sampling, and testing of deep and shallow boreholes and SCPT soundings.

Bedrock was encountered at a depth of 400 ft bgs in borehole DB-02 at Site 3A. The McNairy Formation was encountered overlying bedrock to a depth of 155 ft bgs, for a total thickness of 245 ft. The

Porters Creek Clay was encountered overlying the McNairy to a depth varying between 30 and 60 ft bgs. Terrace Deposits typically overly the Porters Creek Clay to a depth of 15 to 20 ft bgs. Surficial loess deposits were encountered overlying the Terrace Deposits.

Results of settlement calculations predict that the total settlement of a fill constructed to a height of 102 ft above ground surface would result in more than 5 ft of settlement in the center of the disposal cell area. Differential settlement may be as large as 2 to 3 ft across the disposal cell. Detailed design would need to account for such differential settlement by increasing the slopes of the base grades, bottom liner, and drain lines, and by selecting appropriate construction materials. It should be noted that the amount of disposal cell settlement may be overestimated due to difficulties retrieving undisturbed samples in the Porters Creek Clay. Settlement would occur relatively rapidly, with 90% of the settlement occurring in less than 2 years of fill placement, so that settlement would be essentially completed by the time the cell is filled.

Results of bearing capacity analysis indicate that the bearing capacity of the foundation soils is adequate to support a potential CERCLA waste disposal facility at Site 3A.

8.4 SEISMIC DESIGN MODEL

A seismic design model was developed for Site 3A to answer Questions 6 and 7; namely, to determine the PGA and design ground motions for a potential CERCLA waste disposal facility. The model was developed based upon the data collected during the site-specific Fault Study and Geotechnical Study.

A probabilistic seismic hazard analysis was performed to determine what the PGA and other related ground motions (ground shaking frequency, velocity, and displacements) would be at Site 3A for an earthquake having a 2500-year return period. The corresponding PGA value at the top of rock (400 ft bgs) was determined to be 0.71 g.

A site-specific soil amplification factor was calculated for Site 3A based on the shear-wave velocities measured in the deep borehole (DB-02) and SCPT soundings using the methodology employed by REI in its 1999 study. The soil amplification factor for a top-of-rock PGA of 0.71 g was calculated to be 0.67 g. This results in a top-of-soil PGA of 0.48 g for a 2500-year return period earthquake at Site 3A. This value is equal to the top-of-soil PGA value of 0.48 g interpolated from REI in a previous re-evaluation study at PGDP. Therefore, the recommended top-of-soil PGA for design of a potential CERCLA waste disposal facility at Site 3A is 0.48 g.

The shear-wave velocities in the soil column at Site 3A are similar to those measured previously at other locations on the DOE property, resulting in similar design ground motions. Therefore, the design ground motions at Site 3A would be the same as those determined by REI. A uniform hazard spectra that relates ground acceleration to the ground shaking frequency is presented in Chap. 7 to define the design ground motions at Site 3A.

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